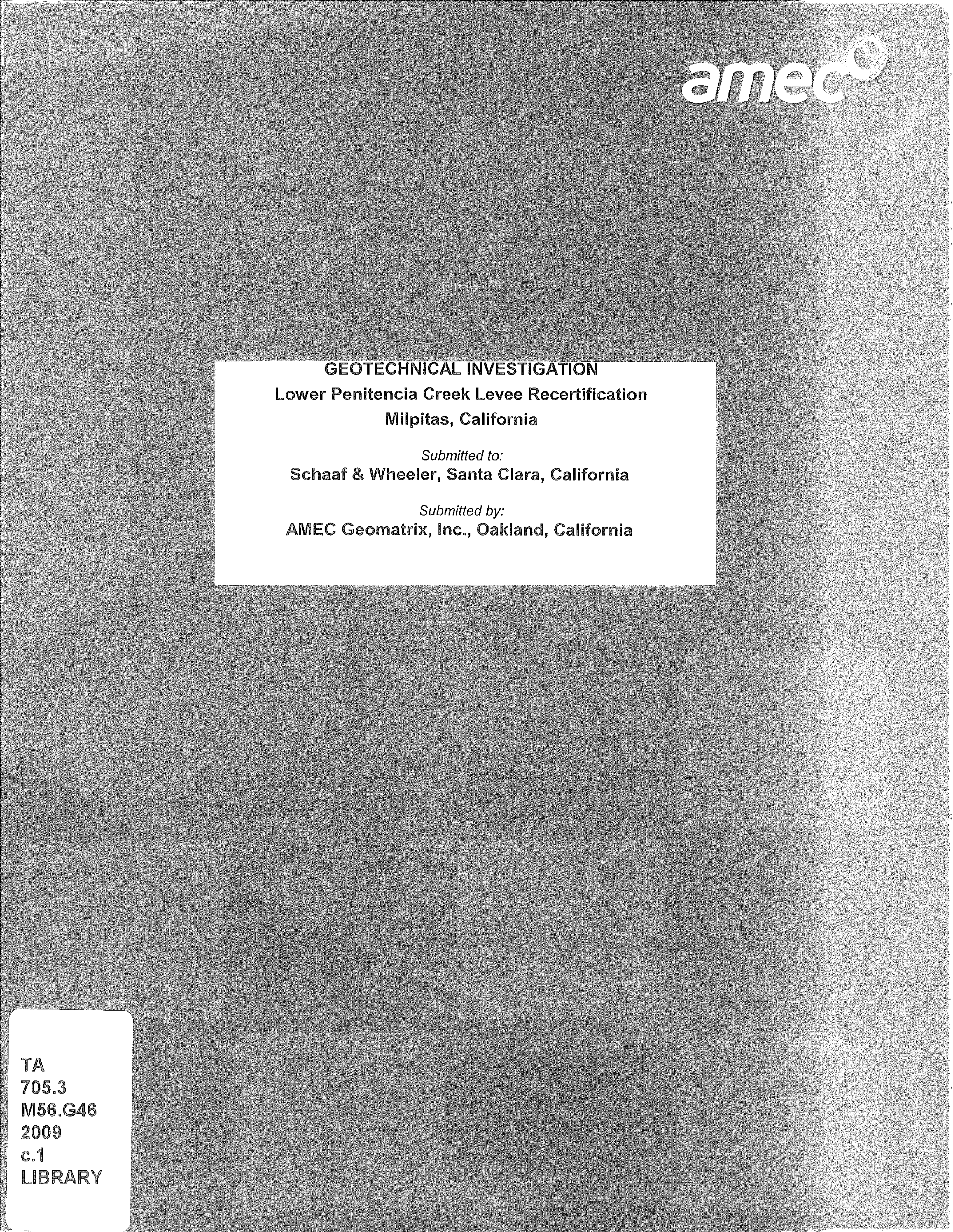

GEOTECHNICAL INVESTIGATION
Lower Penitencia Creek Levee Recertification
Milpitas, California

Submitted to:
Schaaf & Wheeler, Santa Clara, California

Submitted by:
AMEC Geomatrix, Inc., Oakland, California

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Mr. Chuck Anderson
Schaaf & Wheeler, Consulting Civil Engineers
100 N. Winchester Blvd. Suite 200
Santa Clara, CA 95050

Subject: **Geotechnical Investigation Report**
Lower Penitencia Creek Levees
FEMA Levee Recertification
Milpitas, California

Dear Mr. Anderson:

AMEC Geomatrix, Inc. (AMEC) is pleased to submit this Geotechnical Investigation Report to support the Lower Penitencia Creek Levee Recertification project in Milpitas, California. This report was developed in accordance with our Agreement for Professional Services with Schaaf & Wheeler, dated April 15, 2008 (fully executed May 22, 2008). Our investigation included compiling and reviewing existing data, performing a field reconnaissance, performing field exploration, performing geotechnical laboratory testing, performing engineering evaluations and analyses, and preparing this report.

We conclude that we are not aware of any geotechnical considerations that we consider to stand in the way of recertification of the levee in accordance with 44 CFR 65.10.

If you have any questions about this report, please do not hesitate to call any of the undersigned. It has been a pleasure working with you and we look forward to working with you on other future phases of the project.

Sincerely yours,
AMEC Geomatrix, Inc.

James B French

REGISTERED PROFESSIONAL ENGINEER
JAMES B. FRENCH
No. 2018
Exp. 6/30/2010
GEOTECHNICAL
STATE OF CALIFORNIA

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DLR/JBF/LDU
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Enclosure

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AMEC Geomatrix

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GEOTECHNICAL INVESTIGATION REPORT
Lower Penitencia Creek Levees
FEMA Levee Certification
Milpitas, California

1.0 INTRODUCTION

AMEC Geomatrix, Inc. (AMEC, formerly known as Geomatrix Consultants, Inc. [Geomatrix]) is pleased to present the results of our geotechnical investigation for recertification of the Lower Penitencia Creek right (when looking downstream) bank levee, located east of Intestate 880 and between California Circle and Berryessa Creek in the City of Milpitas, California. The location of the project is shown on the attached Site Location Map, Figure 1.

1.1 PURPOSE AND SCOPE

This investigation was performed on behalf of the Santa Clara Valley Water District (the District) and the City of Milpitas. Our work was performed under subcontract to Schaaf & Wheeler, Consulting Civil Engineers (S&W) to evaluate the stability of the levee under existing, steady-state flood, rapid drawdown, and seismic loading, to contain the base flood from a geotechnical standpoint. This study is part of a broader effort to evaluate the levee using the criteria set forth in 44 CFR Section 65.10 of the National Flood Insurance Program (NFIP), which includes minimum certification standards for levee design, operations, and maintenance. These standards must be maintained for the levee to remain accredited by FEMA as providing protection from the estimated 100-year flood. This report strictly addresses the geotechnical aspects of the FEMA levee certification process; other engineering evaluations, such as hydrologic and hydraulic analyses to satisfy the FEMA criteria are not addressed in this report.

The purpose of our work has been to provide geotechnical data, analyses and documents to support a professional opinion about whether the existing levee on the right bank of Lower Penitencia Creek from California Circle to Berryessa Creek in Milpitas meet the requirements of 44 CFR Section 65.10.

The scope of services to accomplish the above-stated purpose was outlined in the Professional Services Agreement (PSA) between AMEC and S&W, dated April 15, 2008 (fully executed May 22, 2008). Tasks performed by AMEC in support of this purpose have included the following:

1) Compiled geotechnical data to describe surface and subsurface conditions, including the engineering properties of levee fill and foundation materials.
2) Performed geotechnical engineering analyses to evaluate liquefaction, seepage, embankment stability, and settlement in general accordance with the United States Army Corps of Engineers (USACE) EM 1110-2-1913. Specific analyses have included the following:
    a) Seepage through and beneath the levee
    b) Levee stability
    c) Levee settlement
3) Prepared this report to present the results of our analyses.

1.2 PROJECT TEAM

The project was directed by S&W as the prime consultant. Under subcontract to S&W, AMEC's work included compiling and reviewing available pertinent geotechnical data, performing field reconnaissance, performing geotechnical engineering analyses, and preparing this report. Geotechnical borings and cone penetration tests (CPTs), as well as geotechnical laboratory tests, were performed by Pacific Geotechnical Engineering (PGE), also under subcontract to S&W; their work was coordinated with S&W and AMEC. PGE's work has been presented under separate cover (PGE 2009), but the boring logs, CPT logs, and laboratory test results are presented in Appendices C and D of this report.

1.3 REPORT ORGANIZATION

The project description, scope of work, and project team are described in Section 1.0. An overview of the levee is presented in Section 2.0. The methods of investigation are described in Section 3.0. The site conditions, including the regional geologic and seismic settings, are presented in Section 4.0. The history of the levee, based on a review Aerial Photographs, is presented in Section 5.0. Surface and subsurface conditions, site geologic setting, groundwater conditions, and seismic site classification are described in Section 6.0. A discussion of our geotechnical analyses is presented in Section 7.0, and our conclusions are presented in Section 8.0. General limitations and references are presented in Sections 9.0 and 10.0, respectively.

Appendices present background data as well as supplemental documentation of our analyses.

2.0 LEVEE DESCRIPTION

As shown on Figure 1, the study area for this project follows Lower Penitencia Creek for approximately 4,000 feet (0.75 miles) between Berryessa Creek to the south and the California

Circle bridge to the north. The study area includes the levee on the right side of the creek (when looking downstream). Immediately upstream of the study area, Lower Penitencia Creek flows through the heavily developed, urban area of the northern Santa Clara Valley. Downstream of the study area, the creek continues for approximately 8 miles through urban areas past existing salt ponds, tidal marshes, and wetlands before reaching the open waters of San Francisco Bay. Tidal influence from the bay extends into the study area.

Within the study area, the right bank levee generally consists of earthen embankments built on natural steam banks. The levee has a concrete lining on the lower slopes and channel bottom from approximately Lower Penitencia Creek (LPC) Station 11+48 to 14+45 and LPC Station 38+10 to Berryessa Creek (BC) Station 2+00. Development to the east of the project levee includes predominantly residential housing. Upstream of the project levee, the levee continues along both Lower Penitencia Creek and Berryessa Creek. Downstream of the project levee, the levee continues after Lower Penitencia Creek joins Coyote Creek just east of the California Circle bridge. The left bank levee between LPC Station 11+48 to 43+75 and BC Station 0+00 to 2+00 was not evaluated under this study.

An original levee along the right bank of Lower Penitencia Creek within the study area was constructed prior to the 1967 air photos. In the late 1980s, the levee was raised by a private developer. In addition to constructing a new levee, the 1980s' improvement project also included extending the concrete lining on the existing concrete lined channel and constructing concrete facing and retaining walls in the locations of electrical towers (Reimer, 1991). Pertinent portions of Reimer (1991) are reproduced in Appendix A.

Aerial photographs of the levee, along with the District's stationing lines, are shown on Figures 2 and 3. The stationing for the study area of Lower Penitencia Creek begins at the confluence of Lower Penitencia and Coyote Creeks, then proceeds along the creek channel upstream approximately 1,100 feet to the beginning of the study area. Stationing continues upstream along Lower Penitencia Creek until Berryessa Creek intersects Lower Penitencia Creek at LPC Station 43+75. At the confluence of Berryessa and Lower Penitencia Creeks, the stationing continues upstream along the Berryessa Creek station line approximately 350 feet. Locations of features described in this report are referenced to the District's levee stationing system; however, our station measurements presented in this report are approximate.

Lower Penitencia Creek enters the southern end of the study area at the confluence of the upper reaches of Lower Penitencia Creek and Berryessa Creek, near BC Station 2+41. Lower Penitencia merges with Berryessa creek near LPC Station 43+75 and BC Station 0+00, then flows north. The creek exits the study area at the point it flows beneath the California Circle Bridge, near LPC Station 11+48.

The crest of the levee is generally gravel-surfaced and serves as an access road for levee maintenance. Between LPC Stations 11+40 to 18+80 and LPC Stations 42+20 to 43+75 and BC Stations 0+00 to 0+90, the crest is asphalt-paved and also serves as a pedestrian path. Between LPC Stations 11+48 to 14+48 and 38+10 to 43+75 and BC Stations 0+00 to 2+41, the creek is lined with concrete along the channel bottom and extending up to approximately the levee toe. PG&E towers are located near LPC Stations 22+00, 29+50, and 37+50. Adjacent to each of these locations, the landside slope has a concrete retaining wall to create a level pad for the tower leg or legs, and the waterside levee slope is concrete-lined for a short reach. A detailed description and photographs of visible, surficial levee features are provided in Appendix B.

Based on available data of cross sections and observations made during our field reconnaissance, existing slopes at the site appear to have maximum heights of approximately 15 feet on the waterside with typical inclinations of about 2:1 (horizontal: vertical). The landside slopes vary from 7 feet in tower locations to 4 feet in locations of concrete lining or transition to an overpass with typical inclinations of about 2.5:1. The concrete-lined portion of the bank just downstream of the confluence with Berryessa Creek is about 1:1. Typical crest widths are approximately 12 feet.

A detailed review of known levee penetrations has been made by S&W in conjunction with this study. A summary of major visible penetrations is provided in Appendix B.

3.0 SITE INVESTIGATION

The site investigation performed for the project consisted of the following tasks:

- Data Review
- Field Reconnaissance
- Field Exploration (performed by PGE)
- Geotechnical Laboratory Testing (performed by PGE)

A summary of the site investigation methods are presented below. More detail on the subsurface exploration and laboratory testing is given in PGE (2009).

3.1 DATA REVIEW

AMEC compiled and reviewed readily available geologic and geotechnical investigation reports concerning the project levee contained in AMEC's project files and in the files of the Santa Clara Valley Water District. The primary information obtained during the data review include previous borings by Terratech for Reimer Associates (Reimer, 1991) shown on a plan and profile sheet for the Lower Penitencia Creek project. Approximate exploration locations identified in previous reports are shown on Figures 2 and 3. Logs of these previous exploration locations and the results of relevant laboratory tests are provided in Appendix A.

In addition to the existing geologic/geotechnical investigation reports, AMEC reviewed published maps and reports to evaluate geologic and geotechnical data relevant to the project site, including readily available geologic and fault maps.

AMEC also reviewed four sets of historical black-and-white aerial photographs with a stereoscope to observe past land use and broad geomorphologic features at and around the project levee for information pertinent to the geologic/geotechnical conditions. Photographs taken in 1967, 1994, and 1998 (2 sets) had scales ranging from 1:350 to 1:3,000, and were supplied by, and were reviewed at the offices of, the Santa Clara Water District. Summaries of the conditions observed on the photographs are provided in Section 5.0.

Detailed citations of these published and unpublished sources are presented in Section 10.0.

3.2 FIELD RECONNAISSANCE

On August 12 and 13, 2008, AMEC performed field reconnaissance of the project levee to observe overall site and levee conditions, including levee geometry, and whether any obvious geologic/geotechnical concerns were exposed.

General reconnaissance activities included the following:

- Visually observing the levee, creek channel, and adjacent areas for evidence of settlement, erosion, ground deformation/cracking, and potential stability problems;
- Approximately measuring levee dimensions and slope inclinations;
- Identifying the presence and condition of erosion protection measures, outfalls, closures, floodwalls, and other structures/improvements along the levee alignments;
- Identifying trees, utilities, and other features that encroach on levee slopes;
- Observing water levels and flow conditions in the creek channel;

- Photographing the levees, adjacent areas, and other features of interest; and
- Locating visible utilities, penetrations, and other features of interest.

Conditions observed during the field reconnaissance were documented in levee field reconnaissance logs, which also include brief descriptions of photographs collected during the reconnaissance effort. Findings of the field reconnaissance, including field logs, photographs, and feature descriptions, in Appendix B of this report.

3.3 FIELD EXPLORATION

A field exploration program, consisting of drilling exploratory borings and performing cone penetration tests (CPTs), was performed by PGE to explore the subsurface conditions on the subject levees. The field exploration program is described in more detail in the following sections.

3.3.1 Exploratory Borings

Three borings (Borings B-1 through B-3) were drilled on the subject levees on October 10, 2008. The exploratory borings B-1 and B-3 were advanced below the top of the maintenance road and B-2 was advanced from the levee crest to depths of approximately 30 to 40 feet at the approximate locations shown on Figures 2 and 3.

The borings were performed under the direction of PGE using mud-rotary drilling techniques. Subsurface materials were sampled using conventional 3-inch outside diameter modified-California (mod-Cal) and 2-inch outside diameter Standard Penetration (SPT) split-spoon samplers driven with a 140-pound auto-trip hammer dropping 30 inches in general conformance with ASTM D1586 procedures. Conventional thin-walled Shelby tube samplers were hydraulically pushed into the ground. The field exploration program is described in more detail in Appendix C. Logs of subsurface conditions encountered in the borings, prepared by PGE, are also presented in Appendix C.

3.3.2 Cone Penetration Tests (CPTs)

Five CPT probes were used to explore the subsurface conditions at the site on October 8, 2008. The CPT probes were advanced by John Sarmiento and Associates, of Orinda, California, working under subcontract to PGE. CPT-02 was advanced from the top of the maintenance road and the other four CPTs were advanced from the levee crest to depths of approximately 24 to 50 feet. The approximate locations of the CPTs are shown on Figures 2 and 3. Graphic logs of CPT measurements are presented in Appendix C.

3.4 GEOTECHNICAL LABORATORY TESTING

Laboratory tests were performed on selected soil samples to evaluate their physical characteristics and engineering properties. Samples were tested for dry density, moisture content, Atterberg limits, grain size distribution, unconsolidated undrained shear strength, consolidated undrained shear strength, and consolidation. The laboratory test program was prepared by PGE in coordination with AMEC. The laboratory tests were performed by PGE and by Cooper Testing Labs of Palo Alto, California, under subcontract to PGE. Tabular and graphic presentations of the test results are presented in Appendix D. Results of select tests are also included on the boring logs in Appendix C.

4.0 GEOLOGIC AND SEISMIC SETTING

4.1 GENERAL

Lower Penitencia Creek originates on the eastern margin of the northern Santa Clara Valley near East Brokaw Road. The creek flows north-northwest to just west of the intersection of Dixon Landing Road and Interstate 880, where it joins Coyote Creek. Coyote Creek then flows west into the southern San Francisco Bay. Calera Creek, Berryessa Creek, and Arroyo de Los Coches are tributaries to Penitencia Creek that drain from the western foothills of the Diablo Range and join the Lower Penitencia Creek south of Milmont Drive.

4.2 REGIONAL GEOLOGY

The San Francisco Bay Area is located at the boundary between the Pacific and North American plates, two large crustal plates that are separated by the north-northwest-trending San Andreas Fault, within the California Coast Ranges Geomorphic Province. The geomorphology of the region includes parts of three prominent, northwest-trending geologic/geomorphic features, which include from west to east the Santa Cruz Mountains, Santa Clara Valley, and the Diablo Range. Santa Clara Valley forms part of an elongated structural block (the San Francisco Bay block) within the central Coast Ranges that contains San Francisco Bay and its surrounding alluvial margins (Page, 1989). This structural block is fault bounded by the San Andreas Fault to the southwest and the Hayward-Calaveras fault zone to the northeast (see Figure 4).

The oldest rocks in the region belong to the Franciscan Complex of Jurassic to Cretaceous age (205 to 65 million years before present [Ma]). These rocks are intensely deformed (i.e., folded, faulted, and fractured) due to ancient tectonic processes and, to a lesser extent, from more recent tectonic processes associated with the San Andreas Fault system. Franciscan rocks generally comprise the "basement" of the Coast Ranges northeast of the San Andreas Fault; Cretaceous granitic rocks, known as the Salinian block, comprise the basement of the ranges located southwest of the San Andreas Fault. A sequence of Tertiary (65 to 1.8 Ma) marine and nonmarine sedimentary rocks unconformably overlies, and locally is in fault contact with the granitic and Franciscan basement rocks in the region.

Quaternary (1.6 Ma to present) surficial deposits are concentrated in the Santa Clara Valley and locally overlie the complexly deformed Cretaceous rocks and Tertiary strata in the adjacent hills. During the Plio-Pleistocene (5 Ma to 11,000 years ago [ka]) epochs, sediments eroded from the uplifting Diablo Range and the Santa Cruz Mountains formed broad alluvial fan complexes along the margins of Santa Clara Valley. The 5 Ma to 300,000-year-old (Plio-Pleistocene) Santa Clara Formation, which consists of a sequence of fluvial and lucustrine sediments, was deposited unconformably on the older Tertiary and Franciscan rocks along the margins of Santa Clara Valley during this time and subsequently has been folded, faulted, and eroded. Although the deposition of the Santa Clara Formation spans a relatively long interval and the stratigraphy of the formation is complex, it has been used as a “marker horizon” for evaluating the recency of faulting in the Santa Clara Valley region. The Santa Clara Formation is unconformably overlain by younger Quaternary and Holocene (11 ka to present) alluvial and fluvial deposits (stream channel, overbank, and flood basin environments), which interfinger to the north with estuarine muds of San Francisco Bay (Helley et al., 1979).

South San Francisco Bay is a north-northwest-trending subsiding basin that is filled primarily with Quaternary alluvium (stream) deposits eroded from the surrounding margins and estuarine (bay mud). The Sangamon and Holocene bay muds are separated by the Quaternary alluvium and eolian (wind blown) sand deposits. Alluvium deposits consist of sediments eroded from the surrounding Santa Cruz Mountains and Diablo Range uplands. These alluvial sediments were transported and deposited by streams and include a mixture of sands, gravels, silts, and clays with highly variable permeability. In contrast, the fine-grained bay muds have very low permeability. The youngest Holocene bay muds underlie almost all of the original San Francisco Bay (Atwater et al., 1977; Helley et al., 1979), but do not extend into the southern portion of the Santa Clara Valley.

Due to movement on the San Andreas and related faults including the Hayward and Calaveras Faults, as well as the previous geologic history, a wide variety of igneous, metamorphic, and sedimentary rocks are present.

Geologic units are classified according to geologic time periods. The most recent time period is the Quaternary Period, which covers from 1.8 million years ago to present. The Quaternary Period is subdivided into the Pleistocene Epoch (1.8 Ma to 11 ka) and the Holocene Epoch (11 ka to present). The Pleistocene includes a number of worldwide glacial periods of low sea level stands. Estuarine (bay) muds have been deposited in San Francisco Bay during high sea level periods of the Sangamon (70 ka to 130 ka) and the Holocene (less than 11 ka) (Atwater et al., 1977).

4.3 SEISMICITY

San Francisco Bay is considered to be one of the more seismically active regions in the world, based on its record of historic earthquakes and its position astride the San Andreas Fault system (see Figure 4). The San Andreas Fault system consists of several major right-lateral strike-slip faults in the region that define the boundary zone between the Pacific and North American tectonic plates. During the past 200 years, faults within this plate boundary zone have produced numerous small-magnitude and at least fifteen moderate to large (i.e., $M > 6$) earthquakes affecting the Bay Area (Toppozada et al., 1981; Ellsworth, 1990; Bakun, 1999). The U.S. Geological Survey (USGS) 2007 Working Group on California Earthquake Probabilities (WGCEP, 2008) estimated an approximately 63 percent probability that at least one major moment magnitude earthquake ($M_W \geq 6.7$) would occur in the San Francisco Bay Area before 2037, with a 21 percent probability that that such an earthquake will occur on the San Andreas Fault.

A large earthquake occurred in June 1838 on the Peninsula segment of the San Andreas Fault (Ellsworth, 1990). Toppozada and Borchardt (1998), Hall et al. (1999) and Hall et al. (2001), and Bakun (1999) reevaluated the data for this earthquake and estimated (local) magnitudes of 7.5, 7.0 to 7.4, and 6.8, respectively. Severe shaking of Modified Mercalli Intensity (MMI) VIII to IX occurred in the vicinity of San Jose during this event, with houses shaken down and damage to walls of adobe structures (Toppozada and Borchardt, 1998). The MMI is an experience-based rating system used to categorize the intensity of shaking (prior to use of the seismograph), and is based on the relative damage done to structures/property and general uneasiness of the population as a result of the earthquake.

An earthquake having an estimated M_L of 6.1 (Bakun, 1999) occurred in the San Jose region on November 26, 1858. This earthquake resulted in cracking of almost every brick, adobe, or concrete building in San Jose (Townley and Allen, 1939), corresponding to MMI VII to VIII (Toppozada et al., 1981).

A strong earthquake occurred on October 8, 1865, and apparently was centered in the Santa Cruz Mountains south of San Jose. This earthquake is estimated to have been of M_L 6.5 (Bakun, 1999), and caused damage to buildings in San Francisco, Santa Clara, San Jose, and other areas (Townley and Allen, 1939). The reported damage from the 1865 earthquake corresponds to MMI VII to IX in the Santa Clara, New Almaden, and San Jose areas (Toppozada et al., 1981).

The 1868 Hayward earthquake occurred north of San Jose along the southern segment of the Hayward Fault and had an estimated magnitude of 7.0 (WGCEP, 2008). Surface rupture apparently extended from Oakland southward to the Warm Springs area of Fremont. The event reportedly resulted in damage and/or the complete destruction of every building in Hayward, as well.as damage to buildings as far south as Gilroy (Townley and Allen, 1939). The reported damage in Santa Clara County corresponds to MMI VII to VIII.

During the M_W 7.8 1906 San Francisco earthquake, the San Andreas Fault ruptured from Shelter Cove near Cape Mendocino southward to near San Juan Bautista. Maximum lateral displacements of 15 to 20 feet occurred north of the Golden Gate at Olema in Marin County (Lawson, 1908). Landslides, liquefaction, and ground settlement occurred throughout the San Francisco Bay region and in the vicinity of the surface rupture as result of this earthquake. Extensive damage occurred to many buildings in San Jose, corresponding to MMI VIII to IX (Toppozada and Parke, 1982). Fissures, round cracks, sand boils, stream bank land sliding, and ground settlement related to the Loma Prieta earthquake were reported along Coyote Creek in the general vicinity (i.e., within several kilometers) of the subject levees by Youd and Hoose (1978).

More recent earthquakes in the region include the 1957 Daly City earthquake on the San Andreas Fault (M_L 5.3); the Coyote Lake and Morgan Hill earthquakes of 1979 and 1984 on the Calaveras Fault (M_L 5.9 and 6.1, respectively); the 1980 Livermore earthquake on the Greenville Fault (M_L 5.8); and the 1989 M_L 7.1 Loma Prieta earthquake on the San Andreas Fault or a parallel subsidiary fault. Of these earthquakes, the strongest shaking and most damage resulted from the October 17, 1989, M_L 7.1 Loma Prieta earthquake.

The 1989 earthquake ruptured on or southwest of the Santa Cruz Mountains segment of the San Andreas Fault and produced MMI VII effects in San Jose. Liquefaction and associated ground-failure effects related to the Loma Prieta earthquake were reported in the vicinity of the subject levees by Knudsen (2000). In the Loma Prieta earthquake liquefaction was observed approximately 5 miles west from the project levees were liquefaction induced settlement was observed. In addition, approximately 7 miles south of the project levees settlement, lateral spreading, and cracks were observed in pavement. The California State Seismic Hazards Zones map of the Milpitas Quadrangle, which covers the subject levees, indicates the subject levees are located in a zone of potential earthquake-induced liquefaction (California Geological Survey, 2001). A slightly more detailed Liquefaction Susceptibility map, published by the USGS and covering the subject levee, indicates that the alluvial fan-estuarine complex deposits (Qhfe) (see Section 6.1) that underlie approximately the downstream half of the project levee have high liquefaction susceptibility, and the alluvial fan deposits (Qhff) (see Section 6.1) that underlie approximately the upstream half of the project levee have moderate liquefaction susceptibility (Witter, et al., 2006, Sheet 2).

5.0 AERIAL PHOTOGRAPH REVIEW

A summary of the levee history, as interpreted from our aerial photograph review, is presented below. It should be noted that each summary only presents significant changes from each prior photograph date.

5.1 12/22/1967 PHOTOGRAPHS

This set of photographs (scale not known) provides complete stereo coverage. A steep sided levee is present and appears to decrease in height toward the southeast.

5.2 9/19/1994 PHOTOGRAPHS

This set of photographs at a scale of ≈ 1 inch = 3000 feet provides complete stereo coverage. Conditions appear to be similar to current existing conditions. A typical cross section across Lower Penitencia Creek (looking downstream) consists of a levee, open channel, marsh, open channel, bench/road, and levee (crest paved?). South of Milmont Drive only one channel is present. Two transmission towers are present on the landside slope of the subject levee at approximately Stations 22+50 and 29+00. Possible concrete slope protection is present on the waterside levee slope opposite the towers. Possible concrete ramps are present along the levee approximately at Stations 38+00 and 2+00, and the bench may be paved in this area.

5.3 8/12/1998 PHOTOGRAPHS

This set of photographs at a scale of ≈ 1 inch = 350 feet provides partial stereo coverage. Conditions appear to be similar to current existing conditions.

5.4 9/28/1998 PHOTOGRAPHS

Only one photograph at a scale of ≈ 1 inch = 350 was available. Conditions appear similar to current existing conditions.

6.0 LEVEE FILL AND FOUNDATION CONDITIONS

6.1 SITE GEOLOGIC CONDITIONS

A recent geologic map covering the subject levees by Dibblee (2005) indicate that Lower Penitencia Creek, including the vicinity of the subject levees, is mapped as underlain by

Holocene alluvial deposits described as gravel, sand, and clay (Qa) representing alluvial fan deposits.

A surficial deposits map covering the area of the subject levees by Helley and Wesling (1989) indicates that the subject levees are underlain by Holocene flood plain deposits (Qhfp). A recent USGS surficial deposits map covering the area of the subject levees by Witter et al. (2006, Sheet 1) indicates that the downstream approximately half of the subject levees are underlain by Holocene alluvial fan-estuarine complex deposits (Qhfe), and the upstream approximately half of the subject levees are underlain by Holocene alluvial deposits (Qhff).

6.2 SUBSURFACE CONDITIONS

The results of the current and past subsurface exploration and laboratory test programs, together with the published geology and information gathered from the air photo review and site reconnaissance were used to develop subsurface stratigraphy and geotechnical properties for use in levee seepage, stability, and settlement analyses. The geotechnical materials encountered in the explorations consisted of Levee Fill over clayey alluvium underlain by sands and gravels. Each material is described below. The material properties uses for our analyses are discussed in Section 7.1.

A profile showing the levee crest elevations and interpreted subsurface stratigraphy is presented in Figure 5.

6.2.1 Levee Fill

The thickness of levee fill encountered in the explorations is approximately 3 to 10 feet depending on the reach cross section location. The fill layer generally consisted of very stiff lean clay (CL) and fat clay (CH), with variable amounts of fine to medium sand.

6.2.2 Upper Clay

Upper native foundation soils generally consisted of a clayey alluvial layer directly beneath the fill. Thickness of the upper clay soil ranges from approximately 10 to 25 feet. This layer consisted of generally very stiff to hard lean clay (CL) and fat clay (CH) with scattered thin (several inches to a foot thick) zones of clayey sand.

6.2.3 Sand

Sand was encountered underlying the upper clay layer in Reaches A and B. In Reach C the sand below the lower clay layer is described in Section 6.2.6 below. This layer consisted of loose to medium dense poorly graded sands with silt. The typical thickness of the sand layer varied from 10 to 18 feet.

6.2.4 Silt

A silt layer was encountered in Reach C above the soft to medium stiff clay layer. This layer consisted of medium dense non-plastic silt material with lean clay.

6.2.5 Soft to Medium Stiff Clay

Based on interpretation of the CPT soil behavior types, CPT-05 in Reach C encountered a 4 feet thick zone of soft to medium stiff clay underlying the upper clay. This clay layer had a lower undrained shear strength than was encountered in other CPTs or in any borings.

6.2.6 Lower Clay

A lower clay layer was encountered underlying the sand layer in Reaches A and B and the upper clay layer in Reach C. The lower clay layer generally consisted of firm to stiff, low to medium plasticity clay with variable amounts of sand and gravel (CL).

6.3 GROUNDWATER CONDITIONS

Groundwater levels could not be observed in borings B-1 through B-3 because mud rotary drilling methods were used. From a previous boring drilled by Terratech in 1982 (Reimer 1991), the depth of the groundwater below ground surface was 9 feet 5½ hours after drilling. This depth corresponds to an elevation of approximately 4 feet (NAVD 88). The groundwater observations in the boring may have been made before the passage of sufficient time to allow groundwater conditions to come to equilibrium. In addition, fluctuations in the groundwater level may occur due to variations in rainfall, temperature, and other factors after the observations were made.

Apparent groundwater levels in the CPT soundings ranged from approximately 11 to 16 feet below the levee crest corresponding to an elevation of approximately 3 to 7 feet (NAVD 88) on October 8, 2008.

Approximate measured and inferred groundwater levels are shown on the levee profile, Figure 5.

7.0 GEOTECHNICAL ANALYSES

The subject levees were evaluated from a geotechnical standpoint in general conformance with the guidelines presented in 44 CFR Section 65.10. The following analyses were performed on critical cross sections thought to represent the worst case seepage and/or slope stability conditions along the levee alignment:

1) The potential for liquefaction triggering was assessed for the embankment fill and underlying alluvial soil deposits during a 475-year return period earthquake event. These analyses were performed assuming typical groundwater conditions (i.e., groundwater located at about the normal year-round water level and not elevated by flooding) due to the low probability of the 475-year earthquake event occurring concurrently with periods of high water.

2) Seepage conditions through and under the levee embankments were evaluated using the finite element computer software SEEP/W (GeoSlope, 2007). Steady-state seepage conditions were conservatively assumed to represent phreatic surfaces through the levee during flood water level conditions (i.e. the analysis was performed as if the flood was in-place for an infinite period of time). Due to the relatively short term duration of the 100-year flood event anticipated along Lower Penitencia Creek and the relatively lower permeability of levee fill materials, a steady-state condition is considered very unlikely to develop in the project levees.

3) Transient seepage conditions were evaluated in Cross Section C using SEEP/W (GeoSlope, 2007) to estimate the amount of seepage expected in the levee during a 100-year flood event. The 100-year hydrograph for this cross section, provided by S&W, was used with 12 incremental time steps of 4 hours each. The permeability of the soil layers was increased by a factor of ten to estimate a more conservative approach to permeability values and the maximum possible seepage into the levee slopes.

4) The stability of the levee slopes was evaluated under both static and earthquake loading conditions using the finite element computer software Slope/W (GeoSlope, 2007). The static stability of the levee slopes was evaluated assuming a range of conditions, including:

    a) As-is conditions with no flood or seismic loading.

    b) Flood loading conditions, with the conservative assumption of steady-state seepage within the levees.

    c) Rapid drawdown, very conservatively assuming fully-developed and un-dissipated pore pressures.

    d) Seismic loading, assuming typical groundwater conditions, which is consistent with the USACE Manual EM 1110-2-1913.

5) Settlement analysis included the evaluation of both long term static settlement and seismically-induced settlement (e.g., seismic compression).

Liquefaction analyses are discussed in Section 7.27.2. Seepage analyses are discussed in Section 7.3. Stability analyses are discussed in Section 7.4. Settlement evaluation is discussed in Section 7.5.

7.1 SELECTION OF CROSS SECTIONS FOR ANALYSES

One representative cross section was selected for each reach. The reaches were defined by the subsurface stratigraphy, cross section geometry, and location along the levee. Reach A is defined by its location along the northern bend of Lower Penitencia Creek between LPC Stations 11+48 to 18+00 and the thick deposit of clay below the levee fill. Reach B is defined by a thick sand deposit, electrical towers, and a straight section of Lower Penitencia Creek through LPC Stations 18+00 to 37+00. Reach C is defined by a concrete lining and silty underlying this reach from LPC Station 37+00 to 43+75 and BC Stations 0+00 to 3+00.

7.1.1 Cross Section Stratigraphy

Our interpreted stratigraphy along the centerline of the levee crest is shown on Figure 5. The stratigraphy of each cross section was developed to be representative of its reach rather than a strict interpretation at the location of the cross section.

The subsurface conditions encountered varied along the length of levee, but were considered to be reasonably consistent within each levee reach selected for analysis. As described in Section 6.2, the subsurface soils encountered during the subsurface exploration program generally consisted of embankment fill underlain by alluvial soil deposits ranging from clay to silt to sand.

7.1.2 Cross Section Geometry

Based on a consideration of surface topography and variable subsurface conditions encountered during the subsurface investigation, three cross sections were selected for analyses. The locations of the analysis cross sections are shown in plan view on Figures 2 and 3, and profile view on Figure 5.

AMEC reviewed the as-built drawings and topographic cross sections provided by S&W at approximately 300-foot intervals along the project alignment. Within each reach, a cross section with the tallest and/or steepest slope was selected; we selected what we considered generally to represent the most conservative geometry with respect to slope stability.

For our analyses, we assumed that the concrete lining that is present on the bottom and side slopes within Reach C served primarily as erosion protection with little structural capacity and little impact on slope stability. This assumption is conservative with regard to slope stability, but reasonable given the concrete is typically thin with a limited amount of reinforcement.

The project team observed the general condition of the exposed PG&E tower footing retaining walls in Reach B and did not observe any indication of distress to these walls. However, we did not perform any analysis to evaluate their adequacy at 100-year flood levels. We also considered the impact that tower footing levee cutouts would have on the levee embankment stability. Assuming that the retaining walls remain stable, we conclude that the presence of levee cutouts result in a net increase in slope stability, due to a reduction in driving forces that exceed any corresponding reduction in shear resistance. If future distress is noted in the retaining walls, they should promptly be evaluated and remedial action implemented as appropriate.

The concrete lining on the waterside slopes located opposite the PG&E tower footings will probably reduce seepage, and will have a corresponding small but positive (if any) impact on slope stability.

7.1.3 Material Properties for Analyses

The primary material properties needed for analysis included unit weights, permeability, and strength. Properties for design are summarized on Table 1. These were selected as follows:

1) Unit weights for predominantly clayey layers were selected based on average values from field and laboratory data and consideration of published typical values. Unit weights for the predominantly sandy layers were selected based on a comparison of published typical values for the material type and calculated values based on laboratory water contents and an assumption of saturated conditions.

2) Permeabilities for clayey materials were selected based on calculations from the published typical values for similar materials, and engineering judgment. The permeability of the silt was selected based on consideration of laboratory consolidation test results and the Kozeny-Carman equations (Carrier, 2003) based on laboratory grain size distributions. Permeabilities for sandy material was selected based on calculations using the Kozeny-Carman equations based on laboratory grain size distributions, published typical values for similar materials, and engineering judgment. In general, values used in analyses were conservatively selected to envelop the upper bound of the data.

3) Strengths for the levee fill material were selected based on consideration of laboratory strength tests, field pocket penetrometer tests, CPT results, and typical correlations with SPT values. In general, values used in for analyses were conservatively selected to envelop the lower bound of the data.

4) Strengths for the native clayey foundation soils were also selected based on a comparison of laboratory strength tests, field pocket penetrometer tests, CPT results, and typical correlations with SPT values. In general, values selected for analyses were conservatively selected to envelop the lower bound of the data.

5) Strengths for the native sandy foundation soils were selected based on a comparison of laboratory results, and typical correlations with CPT and SPT values. In general, values selected for analyses were conservative.

6) Unsaturated strengths for the levee fill and underlying foundation soils anticipated to be in an unsaturated condition during the analysis condition were selected based on an estimate of the soil-water characteristic curve. The soil-water characteristic curve was estimated by soil type and saturated volumetric water content in general conformance with the

procedure outlined by Vanapalli et al. (1996) and described in the SLOPE/W manual (GeoSlope, 2008).

The cross sections are presented with analysis results in Figures 6 through 11. The locations of the cross sections used for analyses are shown in Figures 2 and 3 in plan view and Figure 5 in profile. The cross sections are oriented looking downstream. In general, the crest width was about 12 feet, the embankment heights were typically about 10 feet for the waterside of the levee and range from 7 to 4 feet for the landside of the levee, and the maximum slope inclinations were about 2:1 (horizontal to vertical). However, in the area of the concrete lining, the lower portion of slopes had an approximate slope inclination of 1:1.

7.2 LIQUEFACTION ASSESSMENT

Liquefaction is a soil behavior phenomenon in which a soil located below the groundwater surface loses a substantial amount of strength due to high excess pore-water pressure generated and accumulated during strong earthquake ground shaking. During earthquake ground shaking, induced cyclic shear creates a tendency in most loose to medium dense soils to densify by rearrangement of the soil-particle structure. The potential for excess pore-water pressure generation and strength loss associated with this volume change tendency is highly dependent on the density of the soil, with greater potential in looser soils. Recently-deposited (i.e., geologically young) and relatively loose natural soils, and uncompacted or poorly compacted artificial fills located below the groundwater table, are potentially susceptible to liquefaction. Loose sands and silty sands are particularly susceptible. Silty and clayey soils tend to be less susceptible than sandy soils to liquefaction-type behaviors. Even within sandy soils, the presence of finer-grained materials affects susceptibility. Dense natural soils and well-compacted fills have low susceptibility to liquefaction. Clayey soils and bedrock generally are generally not susceptible to liquefaction.

Permanent ground displacements due to lateral spreads or flow slides and differential settlement are commonly considered significant potential hazards associated with liquefaction. These displacement hazards are direct products of the soil behavior phenomena (i.e., high pore water pressure and significant strength reduction) produced by the liquefaction process. Lateral spreads are ground failure phenomena that occur near abrupt topographic features (i.e., free-faces such as creek banks) and on gently sloping ground underlain by liquefied soil. Lateral spreading movements may be on the order of inches to several feet or more and are typically accompanied by surface fissures and slumping.

Earthquake-induced settlement occurs with the dissipation of excess pore pressure generated by ground shaking that, as described above, is associated with the tendency for loose, saturated soils to rearrange into a denser configuration during shaking. Such dissipation will produce volume decreases (termed consolidation or compaction) within the soils that are manifested at the ground surface as settlement.

Saturated granular materials, potentially susceptible to liquefaction from strong earthquake shaking, were encountered beneath the project levees in Reach A and B. Cross Section A encountered a potential liquefaction zone approximately 30 feet below the levee crest with an approximate thickness of 8 feet. Cross Section B also has a liquefaction zone approximately 24 feet below the levee crest with a thickness of approximately 10 feet.

7.2.1 Liquefaction Analysis Methodology

Ground motions (i.e., Peak Ground Acceleration, or PGA) were estimated from the California Geological Survey (CGS) Probabilistic Hazard Assessment Model, (CGS, 2003) for a 475-year return period earthquake.

The deaggregated contribution to the probabilistic seismic hazard was estimated from the U.S. Geological Survey (USGS), 2002 Interactive deaggregations web site.

Liquefaction susceptibility was assessed using the software program CLiq v1.3 (GeoLogismiki, 2006) for the CPT data and Youd, et al. (2001) for SPT data. CLiq takes measured field CPT data and assesses liquefaction potential and post-earthquake vertical settlement given a user-defined earthquake magnitude and PGA. CLiq utilizes the NCEER method (Youd, et al., 2001) along with calibrated procedures for post-earthquake settlements and lateral displacements by Zhang et al. (2002 and 2004).

It may be noted that CPT correlations underestimate the density and strength of sands that are located near to clay layers; this effect is more pronounced in proportion to the softness of the clay and when a relatively thin sand layer is sandwiched between clay layers. CLiq is not able to correct for this “thin layer” effect, so the liquefaction potential, liquefaction-induced settlement, and strength loss for thin sandy layers tend to be over-estimated in the CLiq plots shown in Appendix E.

7.2.2 Results of the Liquefaction Analysis

The PGA for an alluvial site in the vicinity of Lower Penitencia Creek was estimated to be 0.67g, for a 475-year return period (CGS, 2003).

An earthquake magnitude of 6.9, corresponding to the modal magnitude of the deaggregated probabilistic motions (USGS, 2002), was used for liquefaction analysis.

Results of the liquefaction analyses indicate that the sandy soil layers encountered would likely liquefy during an earthquake event with a 475-year return period. In Reach A the estimated liquefiable zone is located approximately 30 to 38 feet below the levee crest. Reach B is estimated to have a liquefiable zone located approximately 24 to 34 feet below the levee crest. A maximum permanent vertical settlement of 3.5 inches of the levee crest (CPT-3) was estimated to result from liquefaction during an earthquake event with a 475-year return period. A summary of vertical settlement from liquefaction for each reach is located in Table 2. Liquefaction analyses output are included in Appendix E.

7.3 SEEPAGE ANALYSIS

Flood levels can cause a large hydraulic head differential, through or under an embankment, resulting in increased seepage uplift forces in the embankment and/or the embankment foundation. Under adverse seepage conditions, if the weight and cohesiveness of the embankment and/or foundation materials is not sufficient to restrain the seepage uplift forces acting on them, the materials have the potential for piping (internal erosion), which could lead to a loss of material and/or strength, thus impacting levee performance. It is therefore necessary to evaluate seepage conditions under the design flood level.

Under typical seasonal flow conditions, the water level in Lower Penitencia Creek flows in a pair of channels below the toe of the levee (in areas without concrete lining) or a single channel (in areas with concrete lining). During flood conditions, water rises against the waterside of the levees. During a 100-year flood the waterside slope is expected to experience water as high as about 4 to 6 feet below the levee crest. The water surface elevations provided by S&W are approximately equal to the elevation of the landside toe, therefore through-seepage or under seepage is not considered to be a major concern for the Lower Penitencia Creek levee. The water level provided by S&W for a 100-year flood is shown in Figure 5.

Three representative 100-year flood hydrographs are presented in Appendix F. As can be seen in these hydrographs, the duration of high water is anticipated to be about 6 to 12 hours. Based on the relatively short duration of the flood stage and the permeability of the levee and channel materials, it is unlikely that seepage will extend more than a few feet into the levee; however, as is common practice for levee evaluations, we preformed stability calculations based on the assumption that flood levels were in-place for sufficient time that steady-state seepage develops, which yields conservative stability results that bound the worst case scenario. The actual impact of seepage on stability is expected to be less than what has been calculated and reported using this conservative approach.

As a simple and conservative evaluation of the potential for internal erosion or piping to occur, it is common to check that the maximum average exit gradient at the landside toe is less than 0.5 in a steady-state condition. The results of this check are discussed below in Section 7.4.2.

The computer program SEEP/W (GeoSlope, 2007) is a two-dimensional, finite-element seepage analysis program. SEEP/W was used to estimate the steady-state pore pressures within and underlying the levee. The resulting pore pressures were imported into SLOPE/W (GeoSlope, 2007) for subsequent slope stability analysis. SEEP/W also calculated steady-state exit gradients discussed below. The SEEP/W model of the surface and subsurface conditions used for seepage analyses of the levee Cross Sections A through C are provided on Figures 6-1 through 6-3, respectively. The methodologies for determining the initial conditions and hydraulic permeability of the various soil layers are explained below.

7.3.1 Initial Conditions

The initial groundwater conditions were estimated from historical boring data and data from the subsurface exploration program conducted for this study. The groundwater elevations vary from about Elevation 3 feet to 7 feet (NAVD 88), or about 0 to 5 feet above the channel invert. For the seepage analyses, the groundwater elevation was assumed to be approximately 4 feet above the channel invert, which appears generally consistent with the surface elevations of water captured in the LIDAR data throughout this section of Lower Penitencia Creek.

Maximum flood stages were provided by S&W and are reproduced in Appendix F. These values are also shown graphically on Figure 5. For our analyses, the flood water level was simplified to be 4 feet below the levee crest, which is within about 0.5 feet to 1 foot higher than elevations provided by S&W.

7.3.2 Boundary Conditions for Seepage Analyses

Permeabilities of soil materials that are within the saturated zone are estimated using various methods described in Section 7.1. For this analysis, general conductivity functions were chosen based on type of soils encountered in the subsurface explorations.

The following boundary conditions were applied to the model for the flood-stage seepage analyses:

- A fixed total head boundary condition, corresponding to the analyzed water surface elevation, was applied along the waterside slope and channel.
- On the landside, a potential seepage face boundary (nodes that allow water to flow out of the model boundary) was applied and extended to the landside boundary. This and

the potential seepage face boundary allowed SEEP/W to estimate a steady-state phreatic surface through the levee.

- A no-flow boundary was applied on the bottom of the model and vertically along the centerline of the channel.
- The landside vertical drawing extents had a total head boundary condition equal to the elevation of the landside levee toe. This boundary condition conservatively assumed phreatic surface to be at the ground surface, allowing water to flow through the levee and daylight near the landside toe.

Results of the SEEP/W analyses indicate relatively low total exit gradients (0.2 or less) for the 100-year flood levels and steady-state seepage conditions. Results of the steady-state seepage analyses are shown in graphical form in Figures 6-1 through 6-3. Table 2 summarizes the approximate maximum total exit gradients from the seepage analyses.

7.3.3 Permeability

During the subsurface exploration program, soil samples of the levee fill and alluvial soils were retrieved; grain size distribution and consolidation tests were performed on select samples.

From the grain-size distribution of the sandy material, the vertical permeability, k_v, was estimated using the Kozeny-Carman equations (Carrier, 2003). From the higher bound average of all sand grain size distributions for this layer the estimated saturated vertical permeability is about 5×10^{-3} cm/s. This value is on the high end of the range of values calculated and hence is conservative considering the variability in the soil types encountered during the field exploration program. Due to depositional anisotropy, we assumed the horizontal permeability (k_h) to be 20×10^{-3} cm/s (i.e., $k_h / k_v \sim 4$).

Permeability of silt was also calculated using the coefficient of consolidation c_v from consolidation test results using the logarithm of time procedure by Casagrande and the square-root of time procedure by Taylor. Permeabilities were calculated using both procedures and the results were compared with the results from the Kozeny-Carman equations. These calculations suggest an average saturated vertical permeability, k_v, on the order of 1×10^{-4} cm/s for the silt layer. We assumed a ratio of $k_h / k_v \sim 1$ for the alluvial silt (DWR, 2008).

Levee fill materials consisted of soil types and generally ranged from very fine sand to a mixture of sand, silt, and clay with stratified clay deposits. From Holtz and Kovacs (Holtz, 1981), a vertical permeability of 1.0×10^{-5} cm/s was assumed. Based on typical fill placement practices a ratio of $k_h / k_v \sim 4$ for the embankment fill (DWR, 2008).

The upper and lower clay were assumed to be stratified clay deposits. From Holtz and Kovacs (Holtz, 1981), a vertical permeability of $1.0\text{x}10^{-5}$ cm/s was assumed. Based on the microstructure of this material, an assumed ratio of $k_h / k_v \sim 10$ was used for the alluvial clays (DWR, 2008).

7.3.4 Steady-State Seepage Analysis Results

Figures 6-1 through 6-3 show the contours of pressure head output from SEEP/W for Cross Sections A through C. The maximum total gradient for each cross section was taken from the largest contour near the landside toe.

The maximum gradient calculated from the 100-year flood levels assuming steady-state conditions for each section is summarized in Table 2. The 100-year flood water surface elevation is relatively equal to the landside toe elevation. Results indicate that the average total exit gradient is below 0.5 for all reaches.

At tower locations, where the landside toe is lower than the typical cross section of the project levees, the calculated seepage exit gradient is a greater than at the typical cross sections when the concrete facing is neglected. However, when considering the concrete facing of the electrical tower locations, the seepage is comparable to the exit gradient of the typical cross sections.

7.3.5 Transient Seepage Analysis

Steady-state seepage is commonly performed as a conservative upper limit on seepage potential. However, for the Lower Penitencia Creek Levee the duration of a 100-year flood is expected to be on the order of about 36 hours, and the likelihood of steady-state seepage developing is unlikely. To check the degree of conservatism in the steady-state analysis, we performed one transient seepage analysis (Cross Section C). The influence of the concrete channel lining was ignored.

As flood waters rise in the channel, water pressures on the faces of the levees will increase and seepage will progress into the levee. Given enough time, the seepage will pass through the levee and may exit near the toe. Depending on how long the flood water level is elevated, seepage may or may not proceed all the way through the levee. Transient analyses can evaluate the rate that seepage progresses through the levees in response to transient flood water levels.

Using the 100-year water surface elevation hydrograph provided by S&W, the channel water levels as a function of time were set as boundary conditions in the levee channel. For conservatism, in this transient analysis, we increased permeabilities by one order of magnitude above the values indicated in Table 1. We applied the hydrograph (shown in Appendix F) to the channel and channel slopes of Cross Section C to evaluate seepage through the levee. As can be seen in Figure 7-2, at 24 hours into the transient analysis, seepage was calculated to have traveled about 10 feet from the inboard toe into the embankment.

This analysis provides further support for the conclusion that seepage is unlikely to emerge from the embankment during a design level flood that only lasts a matter of hours.

7.4 SLOPE STABILITY ANALYSIS

The computer program SLOPE/W (GeoSlope, 2007) utilizes two-dimensional, force and moment limit-equilibrium methods to calculate factors of safety along specified slip surfaces through an embankment. Spencer's method, which satisfies both force and moment equilibrium and is restricted to a constant inter-slice force function, was used for all analyses. By specifying material zones delineated by strength functions, SLOPE/W estimates the normal stresses acting on a potential slip surface to calculate the shear strength along the surface and compares the strength with estimated (gravity-induced) driving stresses to calculate a factor of safety.

The same cross sections and soil profiles selected for the seepage analyses were used for the slope stability analyses. Levee stability analyses were generally performed using the procedure described in USACE EM 1110-2-1913. The following four loading conditions were considered for each cross section:

- Existing Condition
- Steady-State Seepage Condition
- Rapid Drawdown Condition
- Earthquake Condition

Each condition is discussed below.

7.4.1 Existing Condition

The existing condition considers a situation where the water level in the creek are as described in Section 7.0 and ground water level are as described above in Section 6.3. Slope stability analyses for the existing condition were performed using effective stress strength parameters.

In Cross Section C, the channel surface is relatively steep (approximately 1:1), but lined with concrete. The stability analyses ignored the potential strength benefit of the concrete lining, which is conservative; however the resulting factor of safety using effective strengths was low. Additional stability analyses were performed for Cross Section C where unsaturated strengths were accounted for in the fill and upper clay materials above the phreatic surface. In SLOPE/W a suction term was applied to the fill and upper clay layer to account for suction forces.

The stability of both waterside and landside slopes were calculated for each cross section. The resulting factors of safety range from 1.5 to 5.4, as summarized in Table 2. Figures 8-1 through 8-6 show the critical failure surfaces, factor of safety contours, and model configurations for each section.

7.4.2 Steady-State Seepage Condition

The steady-state seepage condition considers a situation where the flood water level is sustained long enough to create steady-state flow through the levee, increasing pore pressures within the levee and foundation materials, thereby decreasing the strength of the levee and levee foundation materials. The steady-state seepage assumption, though consistent with current standard of the practice, is conservative for the levee and levee foundation conditions identified for our study where the 100-year flood hydrograph indicates a relatively short duration flood event.

Locations of phreatic surfaces were taken directly from the results of the steady-state conditions developed in the seepage analysis using the design flood level. As defined in the seepage analysis, the design flood level is approximately at 4 feet below the levee crest. The stability computations were performed using effective stress strength parameters. Due to the addition of water pressure against the waterside slope of the levee during the 100-year flood, the waterside of the levee more stable in this condition than the existing condition discussed Section 7.4.1. Therefore, only the landside slopes of Cross Sections A through C were analyzed for slope stability under steady-state conditions.

The resulting factors of safety are summarized in Table 2. The results indicate that the levee under the 100-year flood level assuming the steady-state seepage condition are stable, with factors of safety ranging from 2.9 to 4.6. Figures 9-1 through 9-3 show the critical failure surfaces, factors of safety contours, and model configurations for each section.

7.4.3 Rapid Drawdown Condition

For rapid drawdown loading condition for Cross Sections A and B, we conservatively considered a situation in which flood water levels are sustained long enough to create steady-state flow through the levee, and then water levels are quickly (i.e., instantaneously) lowered, reducing the buttressing effect of the water loads on the waterside slope, thereby increasing the driving stresses on the potential failure surface. It is conservatively assumed that drawdown occurs very fast so that no drainage occurs in materials with low permeability. Effective stress strength parameters were used.

In Cross Section C, the lower slope was steeper than the effective strength friction angle of the soil within the slope. Because we considered it appropriate to ignore the concrete lining for the stability analyses, we refined our stability analyses of this cross section (1) by assuming that the calculated transient phreatic water levels would be reasonably conservative for this analyses (rather than using the steady-state phreatic conditions that are not likely to ever occur); and (2) by assuming undrained strengths would be present within the soil at any given time.

Only the waterside slope of the levee was analyzed in the rapid drawdown condition because the landside slope of the levee under this condition will be more stable than the steady-state seepage condition discussed in Section 7.4.3.

The resulting factors of safety are summarized in Table 2. The results show that the levee under rapid drawdown condition is stable, with factors of safety ranging from 1.2 to 2.0. Figures 10-1 through 10-3 show the critical failure surface, factor of safety contours, and the model configuration for each cross section.

7.4.4 Earthquake Condition

SLOPE/W provides an option to specify the resisting shear stress along the potential sliding surface when performing a pseudo-static analysis with an applied seismic coefficient. In this case, SLOPE/W first performs an analysis for a particular slip surface without applying the seismic force, and computes the shear resistance at the base of each slice. Next, the shear resistance is kept constant based on the initial confining stress prior to application of the seismic force. Then, the analysis is repeated for the same slip surface, but including the seismic force. This option (where the undrained strength during earthquake loading is specified based on the confining stress prior to the earthquake) was used in this study.

7.4.4.1 *Initial Seismic Screening*

The seismic stability of the levees was evaluated in two stages. For the initial stage we performed a pseudo-static seismic screening in which a seismic coefficient of 0.10g was applied, as suggested by Seed (1979) for a magnitude 8.25 earthquake. We also reduced the undrained strength by 15 percent for soils that were thought to have a potential for seismic softening, as suggested Seed (1979). Seed indicates that a slope subjected to a seismic coefficient of 0.10g and having resulting pseudo-static safety factor of at least 1.00 for reduced strengths will have deformations less than about 3 feet in a magnitude 8.25 earthquake where the crest PGA is about 0.5g.

For this initial screening, computed factors of safety for both waterside and landside of the levee were in the range of 1.11 to 2.69 (see Table 2). These screening cases are presented on Figures 11-1 through 11-6. The pseudo-static screening analyses of slope stability indicate that the slopes of the levees have factors of safety greater than the minimum value 1.00 for reduced strengths. Although these screening results suggest that the permanent displacements induced by the earthquake will not be larger than about 3 feet for Reaches A, B, and C in a magnitude 8.25 earthquake. Reach B was marginally above the typical screening criteria; therefore, additional evaluation was considered warranted as described in the following text.

7.4.4.2 *Seismic Deformation Analysis*

For this second stage of analysis, permanent deformations were estimated using the yield acceleration concept proposed by Newmark (1965) as modified by Makdisi and Seed (1978). The results of these additional runs are presented on Figures 12-1 through 12-6.

The procedure used to estimate permanent deformation included the following steps.

- A yield acceleration, ky, at which a potential sliding surface would develop a factor of safety of 1.0, was determined using limit-equilibrium, pseudo-static slope stability methods. The yield acceleration depends on slope geometry, phreatic surface conditions, undrained shear strength of the slope material (and/or the residual strength of the liquefied zones), and the location of the potential sliding surface. The soil strength properties were reduced by 20% from the static undrained conditions; therefore the yield accelerations found in these analyses are considered to be conservative.

- The peak ground acceleration for a 475-year return period in alluvium was estimated from the CGS Probabilistic Seismic Hazards Map, 2003, to be 0.67g. With relatively low levees such as those along Lower Penitencia Creek, there is very little amplification of ground motion from the foundation to the crest of the levees; conversely, averaged accelerations within a sliding mass will not be much less than the

PGA. Therefore we consider it appropriate to use the PGA directly for the seismic coefficient.

- For a specified potential sliding mass, the seismic coefficient can be compared with the yield acceleration, ky. When the seismic coefficient exceeds the yield acceleration, down slope movement will occur along the direction of the assumed failure plane. The movement will decelerate and will stop after the level of induced acceleration drops below the yield acceleration, and the relative velocity of the sliding mass will drop to zero. The magnitude of the accumulated permanent displacement can be calculated by double-integrating the seismic coefficient time history where it exceeds the yield acceleration. However, to simplify this calculation, it is normal to use simplified design charts developed by Makdisi and Seed (1978) that relate the yield acceleration, the peak horizontal acceleration, and the predicted earthquake magnitude to the anticipated slope deformation.

The yield acceleration, ky, was calculated using limit-equilibrium stability analyses, as described above. The estimated ky values, factors of safety, and expected deformations for based on this analysis for the waterside and landside slopes of Cross Sections A through C are summarized in Table 2. The computed lateral deformations are from about 1 to 48 inches. This means that, under seismic loading such as could occur during a major earthquake on a nearby fault, portions of these slopes may be expected to deform somewhat. This would result in some loss of freeboard, with the magnitude of freeboard loss being on the order of one third to one half of the anticipated lateral deformations. With the predicted loss of freeboard, the levee is still expected to have approximately 2 feet of freeboard for a 100-year flood event. In addition, some longitudinal cracking of the embankment should be expected, and some cross-levee cracking could occur.

Because of the long return period for a major earthquake, it is unlikely that a flood will occur concurrently with a major earthquake. Similarly it is also unlikely that a major flood (i.e., with a long return interval) would occur in the months following the earthquake.

7.5 SETTLEMENT EVALUATION

7.5.1 Static Settlement

Based on the laboratory consolidation tests, the silt layer underlying the site is highly over-consolidated, suggesting that consolidation settlement should largely happen within a short time after construction (e.g., not more than a couple of years). In addition, the recent surveyed elevations of the crest suggest that the current crest elevation is about the same elevation as the design construction grades. Based on our review of the data, it is our interpretation that this means there has been very little settlement since construction activities in the 1990. In our opinion, it is likely that future settlements from static loads will not be greater than past settlements assuming the groundwater elevation remains the same or higher.

7.5.2 Seismic Settlement

As described above in Section 7.2, earthquake-induced settlement is not expected to exceed more than about 1-4 inches due to densification of underlying sand layers.

8.0 CONCLUSIONS

Based on the results of our geotechnical investigation, it is our opinion that the Lower Penitencia Creek Levees meet the geotechnical criteria for stability, seepage, and settlement set forth in 44 CFR Section 65.10 of the National Flood Insurance Program and will remain stable under the loading conditions described in USACE Design Manual EM 1110-21913. A summary of each of the geotechnical criteria is provided below and the analysis results are summarized in Table 2.

8.1 EROSION OF LEVEE EMBANKMENT DURING BASE FLOOD

It is our understanding that the levee experienced relatively high flood levels since construction. We are not aware of any significant erosion caused by those events. In addition, the levee slopes generally consist of clayey soils that are generally well vegetated with grasses so that they are expected to be generally resistant to erosion. Concrete slopes and channel lining seen in some sections of Lower Penitencia Creek prevent erosion of the underlying channel bed and stream bank slopes. During high flood events the concrete slopes near the electrical towers will also prevent erosion of the levee. We recommend that the project levees be inspected on a regular basis for any signs of increased erosion. The levees should also be inspected for signs of erosion following any major flood events.

8.2 LEVEE EMBANKMENT STABILITY

Detailed analyses were performed to evaluate the embankment stability for various loading conditions. The loading conditions considered during this study included: (1) existing conditions; (2) steady-state seepage; (3) earthquake loading from a 475-year event, and (4) rapid drawdown. For each static loading condition, the calculated minimum Factors of Safety met or exceeded the stability criteria as described in USACE Manual EM 1110-2-1913 Design and Construction of Levees.

It should be noted that our analyses assumed current conditions and geometry as described herein. Stability could be reduced if levee conditions change. We therefore recommend that the levees be inspected on a regular basis and following extreme events (such as significant flood events) to observe any potential erosion, animal burrows, cracks in the concrete lining, or other conditions that may indicate potential degradation of the levee.

8.2.1 Seismic Stability and Deformation

As presented in Section 7.4, we anticipate that a major earthquake could result in up to about 1 to 48 inches of horizontal seismic deformation. Due to the conservatisms in our analyses, we anticipate that most of the levee will experience horizontal deformations near the lower end of this range. However, larger movements may occur in localized areas. We also anticipate that the strains would be distributed over a broad shear zone. Based on geometric considerations, we anticipate that the amount of loss of freeboard would be about one third to one half of the lateral deformation, i.e., on the order of 1 to 16 inches. The crest of the levee is currently 4 to 6 feet higher than estimated 100-year flood levels.

It should be noted that seismic deformation estimates are approximate in nature, and actual values could be more or less than the reported values.

8.3 LEVEE THROUGH AND UNDER SEEPAGE

A detailed analysis of the through and under seepage for a 100-year flood event was performed for the three typical levee cross sections. The seepage through the levee is not expected to jeopardize the embankment stability or cause a piping failure during the steady-state condition. In addition, we anticipate that the steady-state condition will not fully develop in the levee based on the results of the transient analysis.

8.4 LEVEE AND FOUNDATION SETTLEMENT

Analyses were performed to evaluate the static and seismic settlement potential of the subject levee. Based on these results, we estimate future static settlement of less than 1 inch. In addition, we estimate approximately 1 to 4 inches of settlement during a 475-year earthquake event due to seismic densification.

Although localized zones of the foundation may be subject to liquefaction, we calculate that the liquefied strengths will generally be similar to the conservative clay strengths we assumed for our analyses, so the stability will not be greatly affected by these local occurrences of liquefaction.

8.5 ENCROACHMENTS/MAINTENANCE

During our field inspections, we observed several minor encroachments, such as minor landscape irrigation controls for vegetation on the landside slopes. The presence of these encroachments on the levee stability is considered to be negligible and was not incorporated into our stability analyses. In our opinion these encroachments will not decrease the calculated Factor of Safety below the stability criteria established in USACE Manual EM 1110-2-1913 Design and Construction of Levees.

As noted in Appendix B, there appeared to be thick sediment throughout much of the Lower Penitencia Creek channel. Drain holes in the concrete lining in Reach C appeared to be buried and/or clogged with sediment. Sediment should be dredged to keep the drain holes operating properly. Also, build up of sediment may prevent flow through the storm drain outfall near Station 26+00. The control gates should be monitored and maintained as appropriate to keep them functioning properly.

We observed vegetation and evidence of animal burrowing (mostly gophers) in a number of locations, as documented in Appendix B. We recommend regularly scheduled maintenance of the project levee to fill these holes and to remove the animals making them.

9.0 LIMITATIONS

In the performance of its professional services, AMEC, its employees, and its agents comply with the standards of care and skill ordinarily exercised by members of our profession practicing in the same or similar localities. No other warranty, either express or implied, is made or intended in connection with the work performed by us, or by the proposal for consulting or other services, or by the furnishing of oral or written reports or findings. In the event that conclusions or recommendations based on data in this report are made by others, such conclusions and recommendations are not our responsibility unless we have been given an opportunity to review, and we concur in writing with, such conclusions or recommendations.

In presenting data generated or collected by others, AMEC does not warrant or in any other way attest to the accuracy or applicability of the data. Locations of previous explorations by others have been approximated based upon figures provided in reproduced reports, and in many cases do not provide sufficient information or detail for accurate or precise locating of the explorations. As such, locations should be considered approximate.

The scope of work for this study did not include an environmental assessment or investigation for the presence of natural or man-made contaminants in the soil, bedrock, or groundwater. No inference of such conditions should be made from information contained in this report.

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TABLES

TABLES

TABLE 1: MATERIAL PROPERTIES

Lower Penitencia Creek Soil Properties

Soil Type	Properties		Existing	Steady State	Drawdown	Seismic	Source
Fill (CH)	Unit Weight, γ_t	117 pcf					Average of all borings in fill material
	Permeability, k_v	$1.0\text{x}10^{-5}$ cm/s ($3.3\text{x}10^{-7}$ ft/s)					Assumed from Fig. 7.6 in Holtz and Kovacs for very fine sand, mixture of sand, silt and clay, stratified clay deposits, considering a higher permeability because the fill is mostly clayey
	k_h/k_v	4					DWR (2008), page 3-4, Table 3-1 Permeability Values
	Undrained Shear Strength, s_u	1500 psf (CH)				X	Estimated from correlation of strength properties from SPT and CPT data
	Total Stress Cohesion, c	--					
	Total Stress Friction Angle, ϕ	--					
	Effective Cohesion, c'	240 psf	X	X	X		Table 5.8 in Duncan and Wright for CH
	Effective Friction Angle, ϕ'	19°	X	X	X		Table 5.8 in Duncan and Wright for CH
Upper Clay (CH)	Unit Weight, γ_t	120 pcf					Average of all borings in upper clay layer
	Permeability, k_v	$1.0\text{x}10^{-6}$ cm/s ($3.28\text{x}10^{-8}$ ft/s)					Assumed from Fig. 7.6 in Holtz and Kovacs for stratified clay deposits
	k_h/k_v	10					DWR (2008), page 3-4, Table 3-1 Permeability Values
	Undrained Shear Strength, s_u	1000 psf				X	Estimated from correlation of strength properties from SPT data and CPT data
	Total Stress Cohesion, c	--					
	Total Stress Friction Angle, ϕ	--					
	Effective Cohesion, c'	50 psf *	X	X	X		Added to prevent circle from walking out of the slope *c' = 100 psf in Cross Section C because slope face 1:1 and concrete ignored
	Effective Friction Angle, ϕ'	28°	X	X	X		Estimated from chart by Ladd (1977) using average PI=37

TABLE 1: MATERIAL PROPERTIES

Lower Penitencia Creek Soil Properties

Soil Type	Properties		Existing	Steady State	Drawdown	Seismic	Source
Sand	Unit Weight, γ_t	124 pcf					Average of all borings in sand layer
	Permeability, k_v	4.53×10^{-3} cm/s (1.49×10^{-4} ft/s)					Kozeny-Carman average value for sand layer
	k_h/k_v	4					DWR (2008), page 3-4, Table 3-1 Permeability Values
	Undrained Shear Strength, s_u	--					
	Total Stress Cohesion, c	0 psf				X	
	Total Stress Friction Angle, ϕ	32°				X	Estimated from correlation of strength properties from blow counts - Meyerhof and from CPT data
	Effective Cohesion, c'	0 psf	X	X	X		
	Effective Friction Angle, ϕ'	32°	X	X	X		Assume same as ϕ
Silt	Unit Weight, γ_t	124 pcf					Average of all borings in soft clay layer
	Permeability, k_v	1.45×10^{-4} cm/s (4.76×10^{-6} ft/s)					Average value from Kozeny-Carman for non-plastic silt and consolidation back calculation
	k_h/k_v	1					DWR (2008), page 3-4, Table 3-1 Permeability Values
	Undrained Shear Strength, s_u	--					
	Total Stress Cohesion, c	400 psf				X	Cu test results in silt layer
	Total Stress Friction Angle, ϕ	12.8°				X	Cu test results in silt layer
	Effective Cohesion, c'	0 psf	X	X	X		Cu test results in silt layer
	Effective Friction Angle, ϕ'	31°	X	X	X		Cu test results in silt layer

TABLE 1: MATERIAL PROPERTIES

Lower Penitencia Creek Soil Properties

Soil Type	Properties		Existing	Steady State	Drawdown	Seismic	Source
Lower Clay	Unit Weight, γ_t	127 pcf					Average of all borings in gravel layer
	Permeability, k_v	$1.0x10^{-6}$ cm/s $(3.28x10^{-8}$ ft/s)					Assumed from Fig. 7.6 in Holtz and Kovacs for stratified clay deposits
	k_h/k_v	10					DWR (2008), page 3-4, Table 3-1 Permeability Values
	Undrained Shear Strength, s_u	1100 psf				X	Estimated from correlation of strength properties from SPT data and CPT data
	Total Stress Cohesion, c	--					
	Total Stress Friction Angle, ϕ	--					
	Effective Cohesion, c'	0 psf	X	X	X		
	Effective Friction Angle, ϕ'	33°	X	X	X		Estimated from chart by Ladd (1977) using average PI=16
Soft to Medium Stiff Clay Cross Section C only	Unit Weight, γ_t	120 pcf					Average of all borings in clay layer
	Permeability, k_v	$1.0x10^{-6}$ cm/s $(3.28x10^{-8}$ ft/s)					Assumed from Fig. 7.6 in Holtz and Kovacs for stratified clay deposits
	k_h/k_v	10					DWR (2008), page 3-4, Table 3-1 Permeability Values
	Undrained Shear Strength, s_u	300				X	Estimated from correlation of strength properties CPT data
	Total Stress Cohesion, c	--					
	Total Stress Friction Angle, ϕ	--					
	Effective Cohesion, c'	100 psf	X	X	X		Table 5.8 in Duncan and Wright
	Effective Friction Angle, ϕ'	29°	X	X	X		Estimated from chart by Mitchell (1976) using average PI=20

TABLE 2: LEVEE ANALYSIS RESULTS

Analysis Condition	Strength Type	Required minimum Factor of Safety	Computed Factor of Safety					
			Reach A		Reach B		Reach C	
			W	L	W	L	W	L
Existing	Drained	1.5	1.88	4.13	1.78	4.20	1.53	5.41
Steady-State Seepage	Drained	1.4[1]	--	3.22	--	4.51	--	2.84
Rapid Drawdown	3-stage	1.2[1]	1.27	--	1.23	--	1.96	--
Pseudo-Static Screening	Undrained	1.0	1.60	2.39	1.11	2.41	1.33	2.69
			Seismic Yield Coefficient					
Seismic Yield Acceleration	Undrained	n/a	1.00 0.18g	1.00 0.25g	1.00 0.12g	1.00 0.25g	1.00 0.19g	1.00 0.30g
			Maximum Exit Gradient					
Maximum Exit Gradient	Steady-State Seepage			0.11		<0.1		0.38
			Range of Seismic Deformation[2] (inches)					
Seismic Deformation (horizontal)			5-29	2-15	9-48	2-14	5-26	1-9
			Range of Seismically Induced Settlements[3] (inches)					
Seismic Settlement			1.0-1.5		2.0-3.5		0.5-1.2	

[1] USACE, 2000

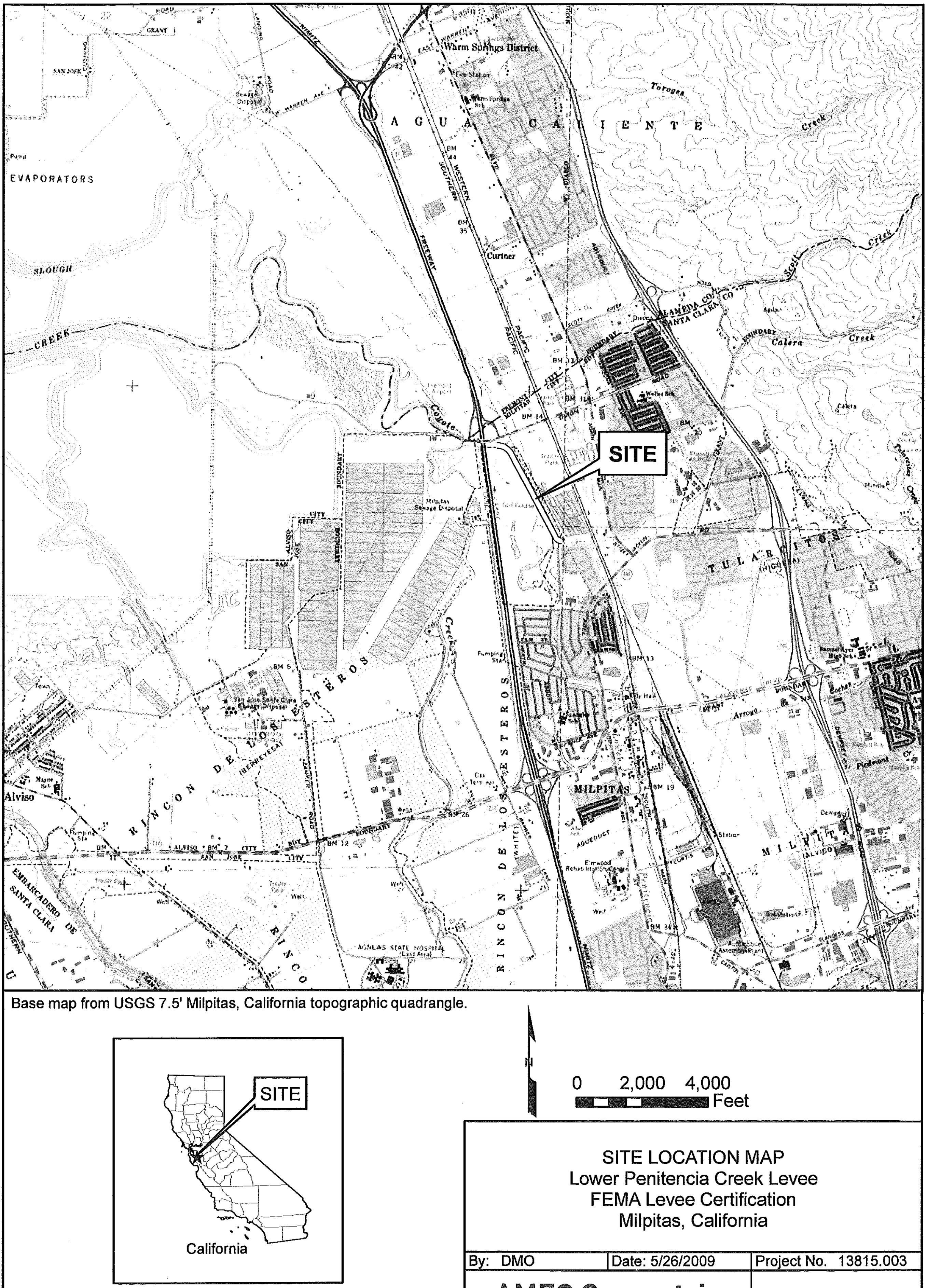
[2] Makdisi and Seed (1978)

[3] CLiq analysis (2006)

W denotes waterside. L denotes landside.

FIGURES

FIGURES



Base map from USGS 7.5' Milpitas, California topographic quadrangle.

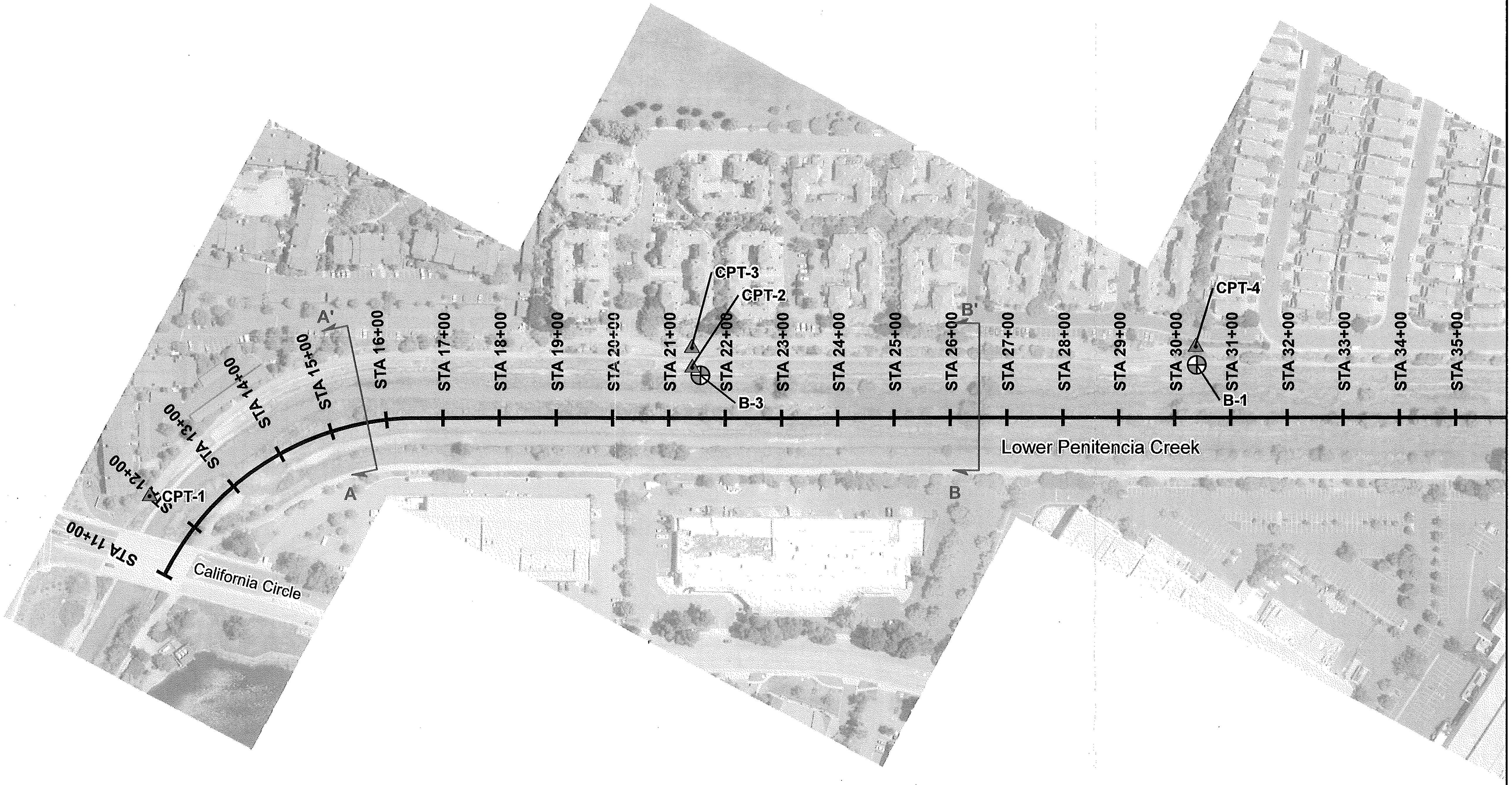
SITE LOCATION MAP
Lower Penitencia Creek Levee
FEMA Levee Certification
Milpitas, California

By: DMO	Date: 5/26/2009	Project No. 13815.003

AMEC Geomatrix

Figure 1

File path: S:\13800\13815\13815.003\task_c03109_0526_lpclr_fig_01.mxd



Explanation

▲ CPT, Pacific Geotechnical Engineering, 2008

⊕ Boring, Pacific Geotechnical Engineering, 2008

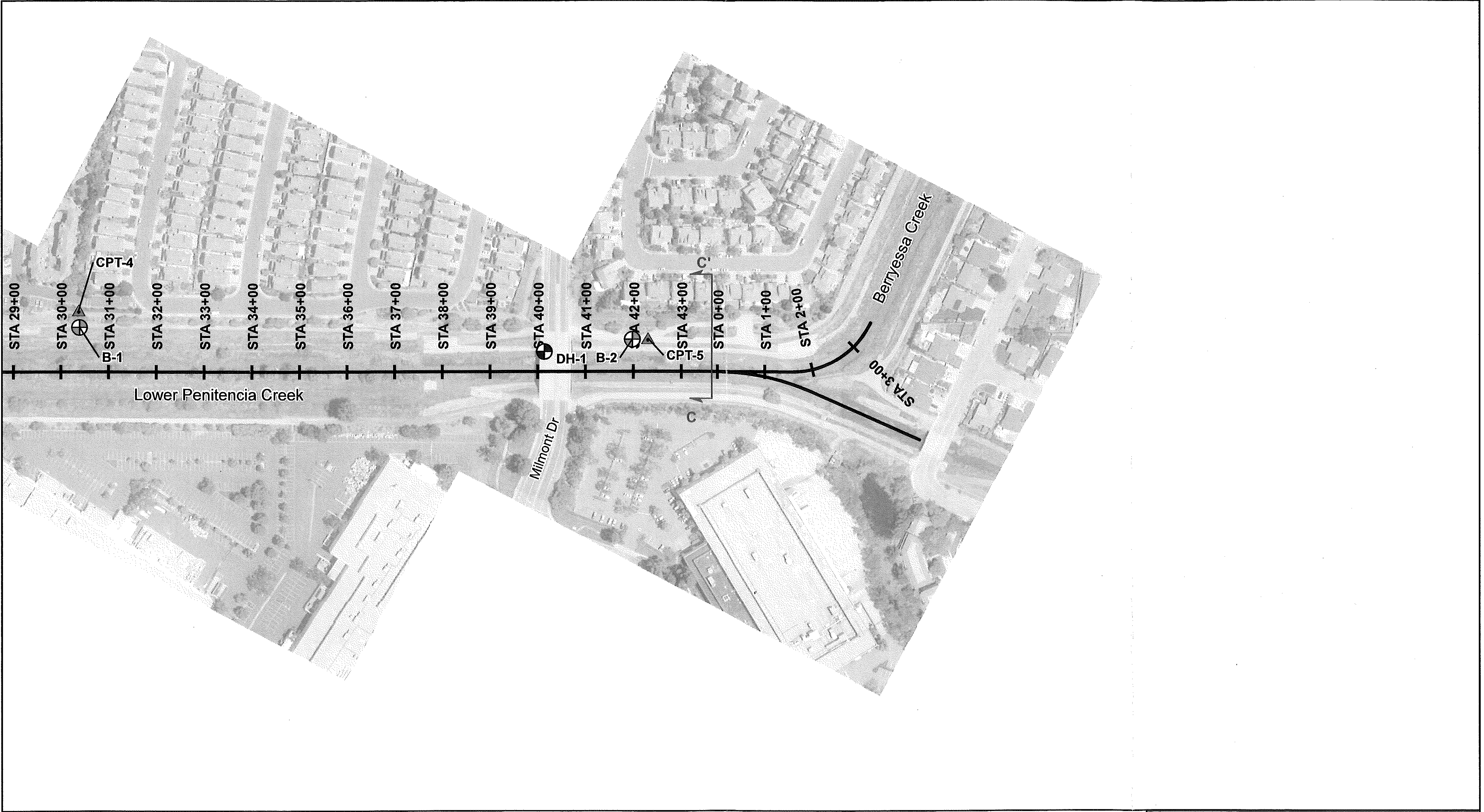
◓ Boring, Terratech, December 1982, for Reimer Associates

0 100 200
Feet

LEVEE ALIGNMENTS AND
FIELD EXPLORATION MAP, NORTHERN REACH
Lower Penitencia Creek Levee
FEMA Levee Certification
Milpitas, California

By: DMO	Date: 6/3/2009	Project No. 13815.003
AMEC Geomatrix		Figure 2

File path: S:\13800\13815\13815.003\task_c03\09_0526_lpclr_fig_02.mxd



Explanation

▲ CPT, Pacific Geotechnical Engineering, 2008

⊕ Boring, Pacific Geotechnical Engineering, 2008

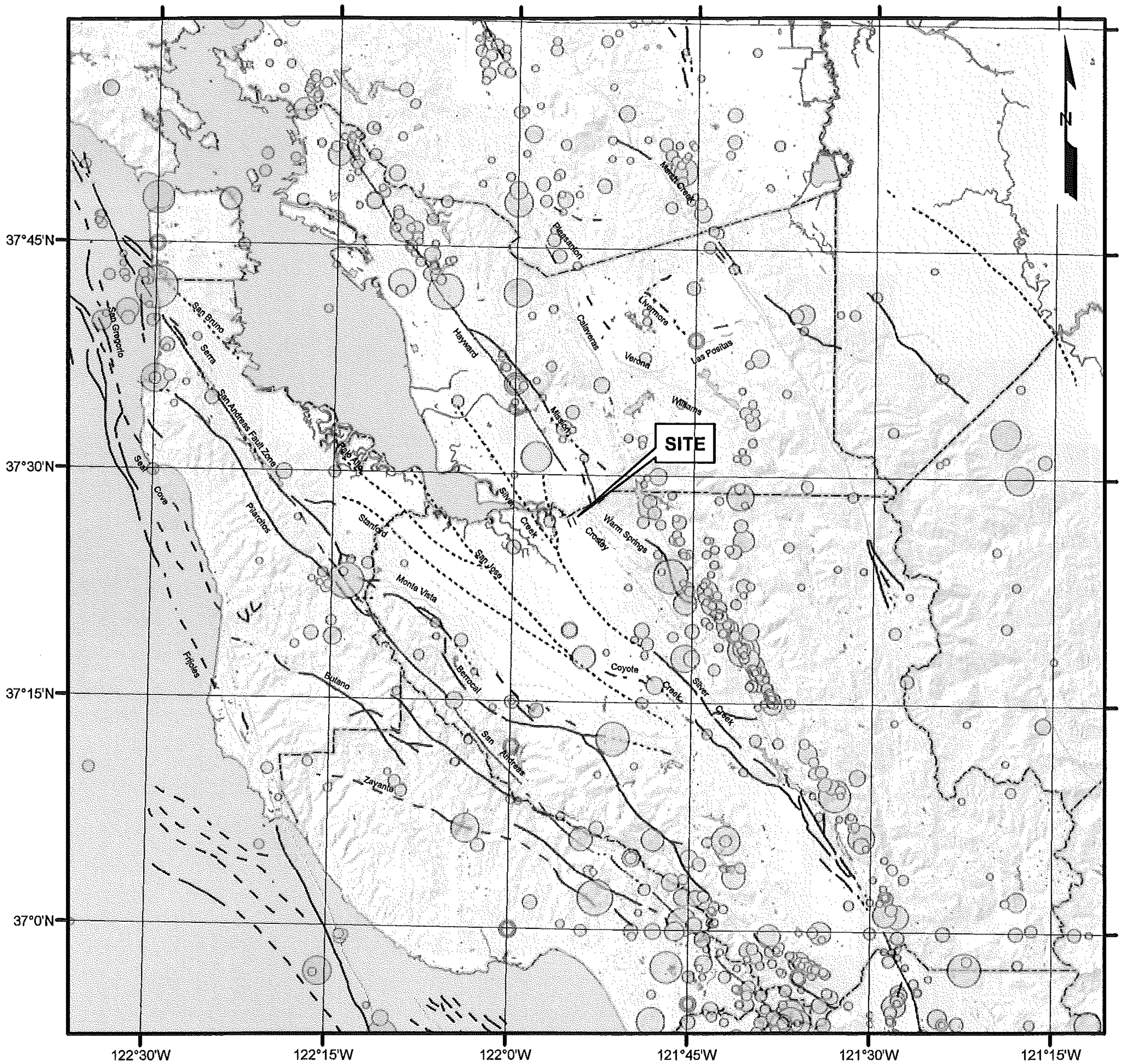
● Boring, Terratech, December 1982, for Reimer Associates

0 100 200
Feet

LEVEE ALIGNMENTS AND
FIELD EXPLORATION MAP, SOUTHERN REACH
Lower Penitencia Creek Levee
FEMA Levee Certification
Milpitas, California

By: DMO	Date: 6/3/2009	Project No. 13815.003
AMEC Geomatrix		Figure 3

File path: S:\13800\13815\13815.003\task_c03\09_0526_lpclr_fig_03.mxd



Explanation

Fault traces on land shown as solid where well located, dashed where uncertain or inferred, dotted where buried. Data from Jennings (1994).

- Faults with historic surface rupture/creep
- Faults that displace Holocene (~11 ka) or latest Pleistocene (~20 Ka) deposits or geomorphic surfaces.
- Faults that displace Late Quaternary (~ 780 ka) deposits or geomorphic surfaces.
- Quaternary faults, undifferentiated (1.8 Ma).

Independant Seismicity (1769-2008)

- ● 3.0
- ● 4.0
- ● 5.0
- ● 6.0
- ● 7.0

0 5 10 Miles

REGIONAL FAULT AND HISTORIC SEISMICITY MAP
Lower Penitenica Creek Levee
FEMA Levee Certification
Milpitas, California

By: DAL	Date: 6/2/2009	Project No. 13815.003
AMEC Geomatrix		Figure 4

File path: S:\13800\13815\13815.003\task_c03\09_0526_lpclr_fig_04.mxd

APPROXIMATE LOCATION OF CALIFORNIA CIRCLE BRIDGE
APPROXIMATE LOCATION OF CROSS SECTION A-A'
APPROXIMATE LOCATION OF ELECTRICAL TOWER
APPROXIMATE LOCATION OF CROSS SECTION B-B'
APPROXIMATE LOCATION OF ELECTRICAL TOWER
APPROXIMATE LOCATION OF ELECTRICAL TOWER
APPROXIMATE LOCATION OF MONTAGUE EXPRESSWAY BRIDGE
APPROXIMATE LOCATION OF CROSS SECTION C-C'
CPT-1
CPT-3
CPT-2
CPT-4
CPT-5
FILL
FILL
FILL
UPPER CLAY
UPPER CLAY
UPPER CLAY
SILT
SILT
SAND
SAND
SAND
SAND
LOWER CLAY
LOWER CLAY
LOWER CLAY
LOWER CLAY
?
TD Elev. -50.50 ft
TD Elev. -51.40 ft
TD Elev. -20.40 ft
TD Elev. -50.50 ft
TD Elev. -44.70 ft
ELEVATION, feet, NAVD 88 Datum
20
10
0
-10
-20
-30
-40
-50
-60
CHANNEL CONCRETE LINING
CHANNEL CONCRETE LINING
REACH A
REACH B
REACH C
15+00
20+00
25+00
30+00
35+00
40+00
LEVEE STATIONING, feet

Surfaces Legend

- CREST ELEVATION
- NATURAL GROUND SURFACE ELEVATION (PROJECTED FROM LANDSIDE TOE)
- WATER SURFACE ELEVATION (PROJECTED FROM CHANNEL)
- CHANNEL INVERT ELEVATION (PROJECTED FROM CHANNEL)

Boring Plot Legend

Blows per Foot
◇ SPT ● Modesol (Modified, 0.65)
□ Water Content, %
├─┤ PL-LL, %
Su (ksf)
+ Pocket Pen ★ UC ▽ Water Level at Time of Drilling
× Torvane ✱ UU ▼ Water Level at End of Drilling

Vertical Scale = 5.00 ft/in.
Horizontal Scale = 100.00 ft/in.

Begin Line North 100.00 East 0.00
End Line North 100.00 East 5000.00

LIST OF BORING NUMBER AND OFFSET

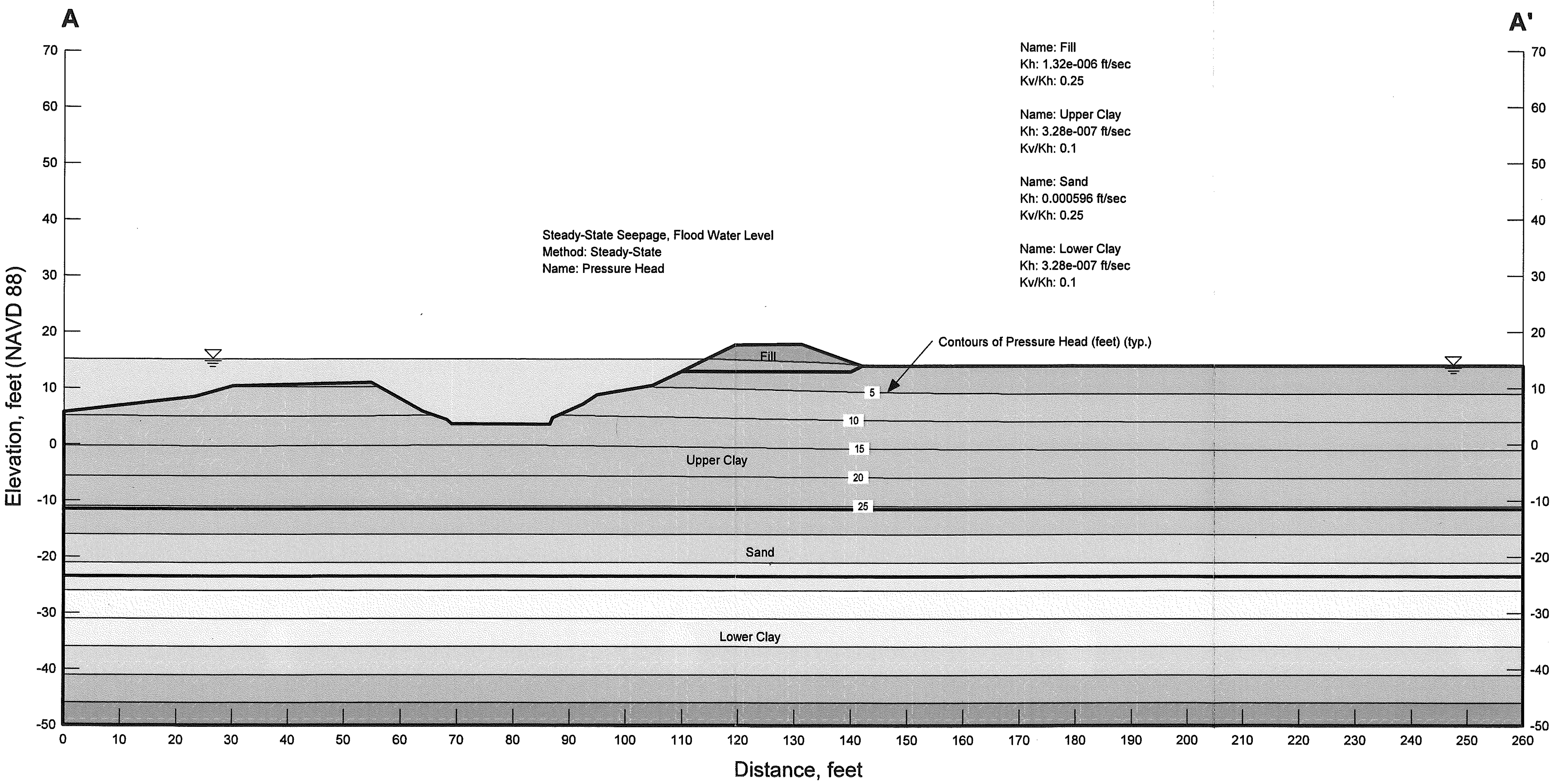
CPT-3	-1 in.	(-100.0 ft)
CPT-2	-0.8 in.	(-80.0 ft)
CPT-5	1.2 in.	(120.0 ft)

NOTES
1. PROFILE ELEVATIONS WERE PROVIDED BY SCHAAFF & WHEELER.
2. ELEVATIONS ARE APPROXIMATE.
3. BORING AND CPT LOCATIONS ARE SHOWN IN FIGURES 2 AND 3. ELEVATIONS OF BORINGS AND CPTS WERE ESTIMATED BASED ON PROFILE ELEVATIONS.
4. GEOLOGIC INTERPRETATIONS ARE GENERALIZED AND CONTACT ELEVATIONS ARE APPROXIMATE AND BASED ON AVAILABLE INFORMATION.

East Bank Levee Profile
Lower Penitencia
FEMA Levee Certification
Milpitas, CA

By: JH | Date: 03/27/09 | Project: 13815.003
AMEC Geomatrix | Figure 5

Directory: I:\Project\13000\13815.003\12000 CALC EVAL ANALYSES\Geo Studio\Section AV_RS_1640_SteadyState.gsz Date: 6/15/2009 Time: 10:23:20 AM Last Edited By: Levulett, Darcie



Steady-State Seepage, Flood Water Level
Method: Steady-State
Name: Pressure Head

Name: Fill
Kh: 1.32e-006 ft/sec
Kv/Kh: 0.25

Name: Upper Clay
Kh: 3.28e-007 ft/sec
Kv/Kh: 0.1

Name: Sand
Kh: 0.000596 ft/sec
Kv/Kh: 0.25

Name: Lower Clay
Kh: 3.28e-007 ft/sec
Kv/Kh: 0.1

amec

Scale
1 inch = 20 feet

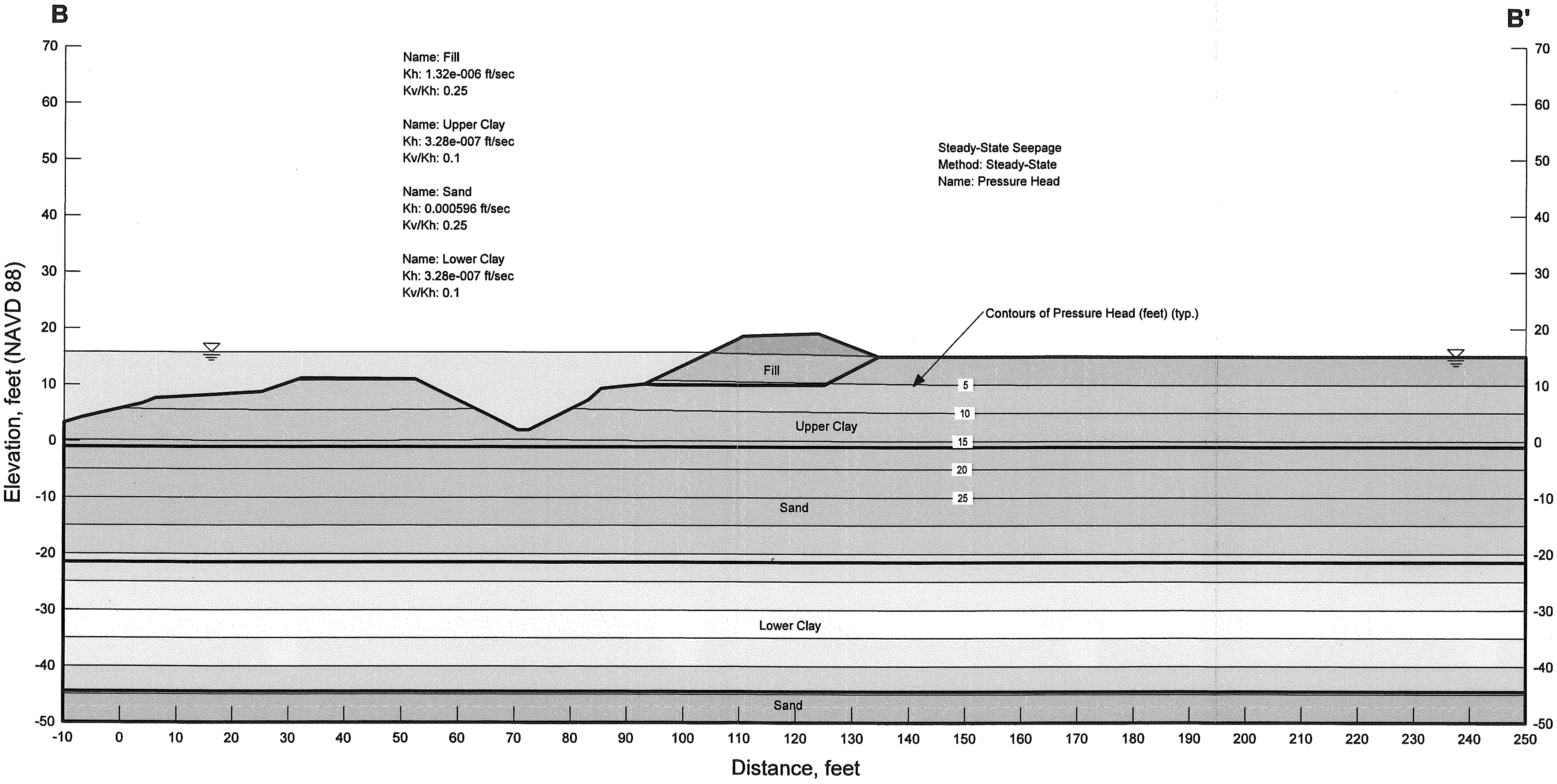
Project Number 13815.003	Drawn D. Levulett
Task 0C.03	Analyzed D. Levulett
Date 6/15/2009	Checked

Lower Penitencia Creek
FEMA Levee Recertification
Milpitas, California

Seepage Analysis Result
Flood Level Water Condition
Cross Section A
Levee Station 15+65

Figure
6-1

Directory: I:\Project\13000\13815.003\12000 CALC EVAL ANALYSES\Geo Studio\Section BIB_RS_3026_SteadyState.gsz Date: 6/15/2009 Time: 10:49:42 AM Last Edited By: Levulett, Darcie



amec

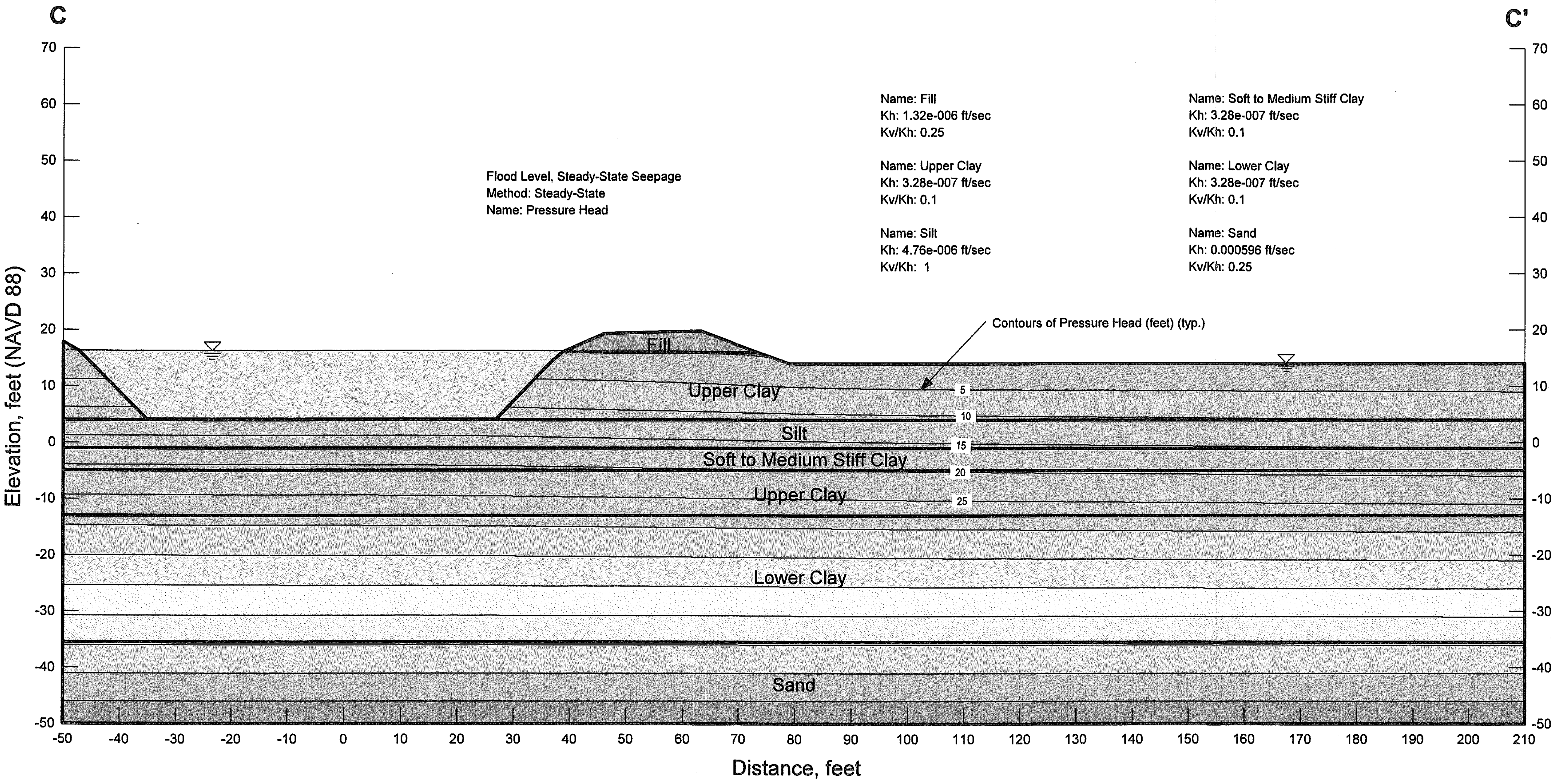
Scale
1 inch = 20 feet

Project Number 13815.003	Drawn D. Levulett
Task 0C.03	Analyzed D. Levulett
Date 6/15/2009	Checked

Lower Penitencia Creek
FEMA Levee Recertification
Milpitas, California

Seepage Analysis Result
Flood Water Condition
Cross Section B
Levee Station 26+51

Figure
6-2



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amec

Scale
1 inch = 20 feet

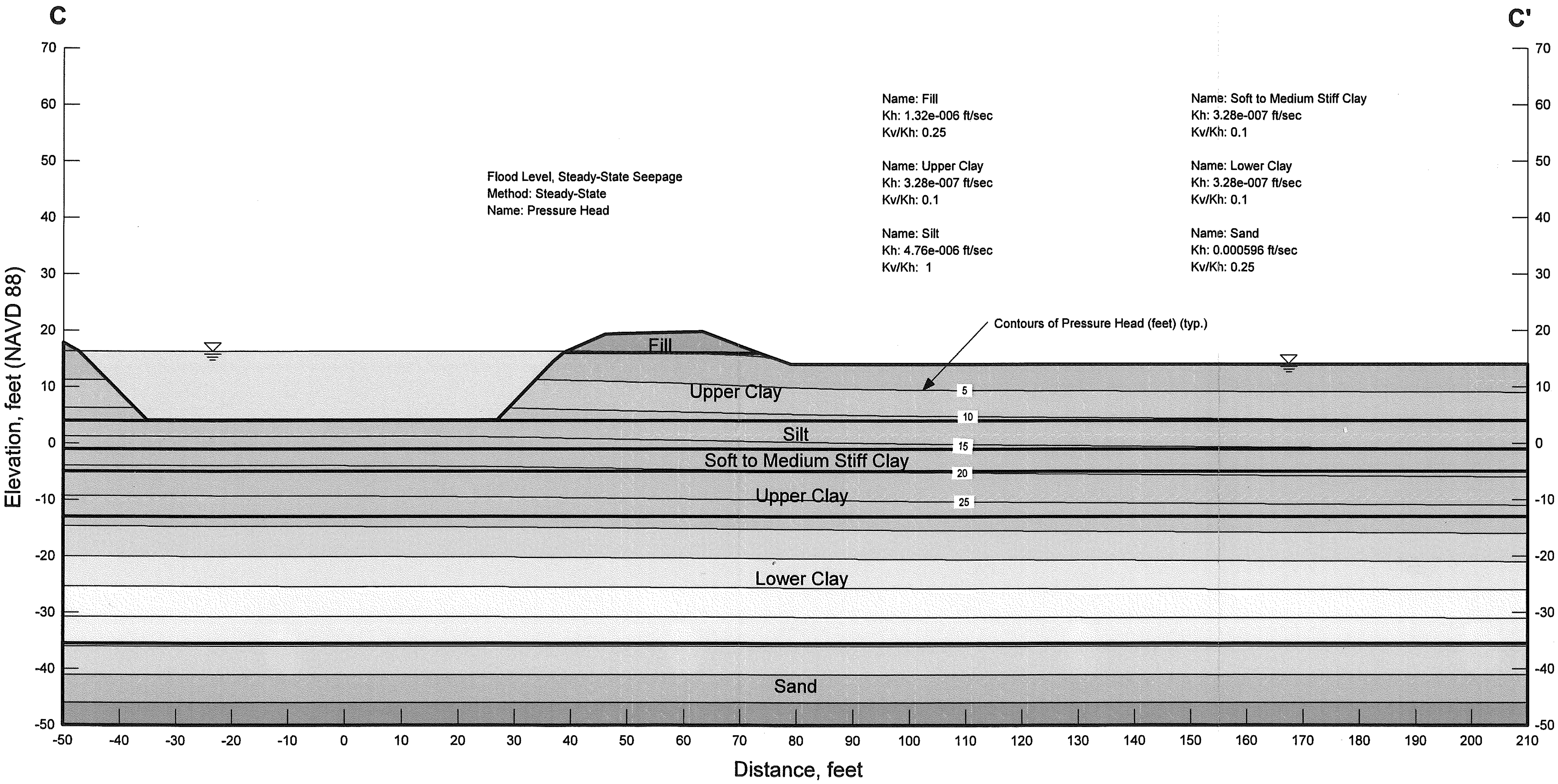
Project Number 13815.003	Drawn D. Levulett
Task 0C.03	Analyzed D. Levulett
Date 6/15/2009	Checked JJJ

Lower Penitencia Creek
FEMA Levee Recertification
Milpitas, California

Seepage Analysis Result
Flood Water Level Condition
Cross Section C
Levee Station 43+65

Figure
6-3

Directory: I:\Project\13000\13815.003\12000 CALC EVAL ANALYSES\Geo Studio\Section C\No Concrete\C_RS_4440_SteadyState.gsz Date: 6/15/2009 Time: 11:19:18 AM Last Edited By: Levulett



Flood Level, Steady-State Seepage
Method: Steady-State
Name: Pressure Head

Name: Fill
Kh: 1.32e-006 ft/sec
Kv/Kh: 0.25

Name: Upper Clay
Kh: 3.28e-007 ft/sec
Kv/Kh: 0.1

Name: Silt
Kh: 4.76e-006 ft/sec
Kv/Kh: 1

Name: Soft to Medium Stiff Clay
Kh: 3.28e-007 ft/sec
Kv/Kh: 0.1

Name: Lower Clay
Kh: 3.28e-007 ft/sec
Kv/Kh: 0.1

Name: Sand
Kh: 0.000596 ft/sec
Kv/Kh: 0.25

Contours of Pressure Head (feet) (typ.)

amec

Scale
1 inch = 20 feet

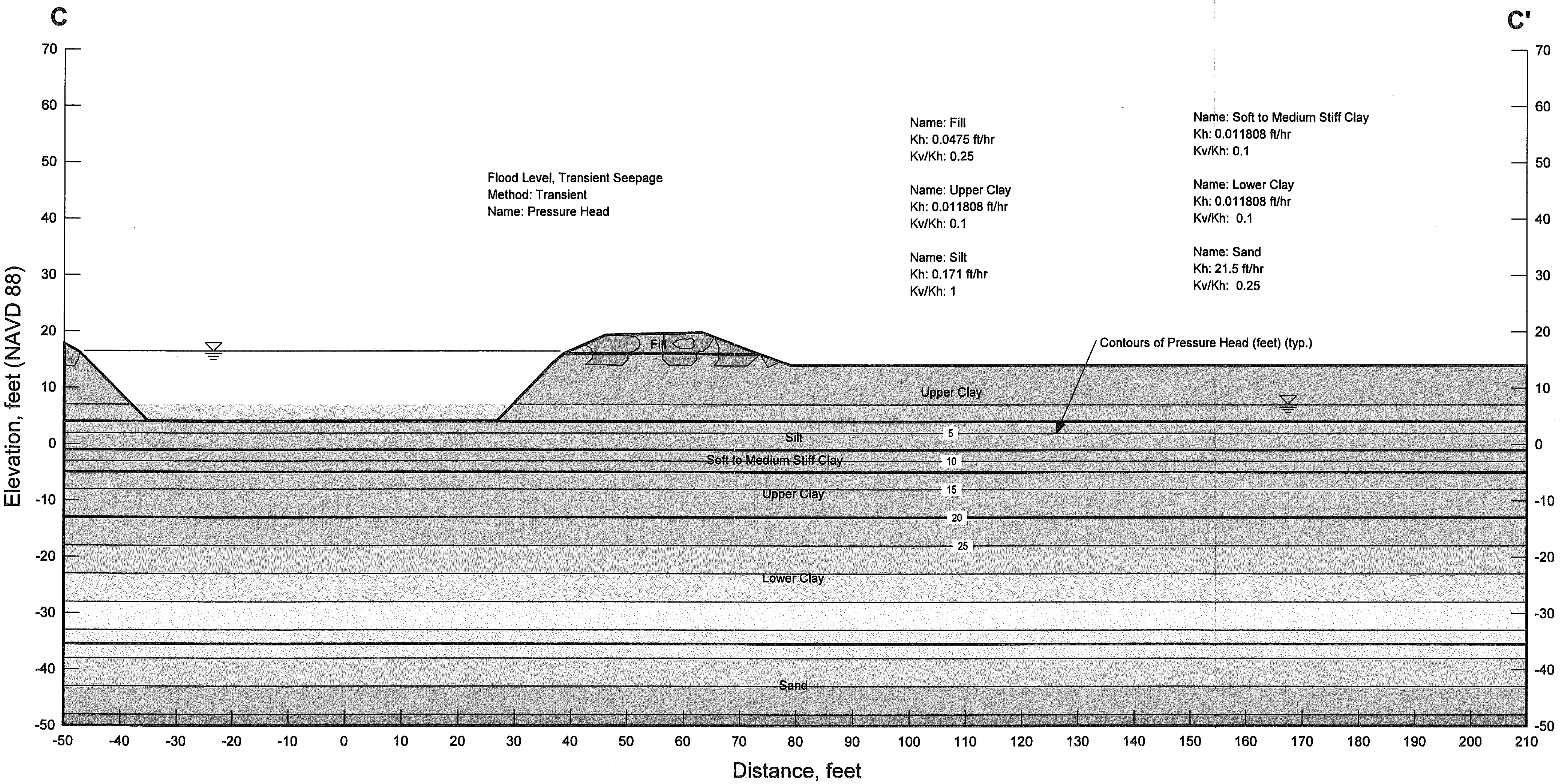
Project Number 13815.003	Drawn D. Levulett
Task 0C.03	Analyzed D. Levulett
Date 6/15/2009	Checked

Lower Penitencia Creek
FEMA Levee Recertification
Milpitas, California

Seepage Analysis Result
Flood Water Level Condition
Cross Section C
Levee Station 43+65

Figure
6-3

Directory: I:\Project\13000\13815.003\12000 CALC EVAL ANALYSES\Geo Studio\Section C\No Concrete\C_RS_4440_Transientx10.gsz Date: 6/9/2009 Time: 10:56:22 AM Last Edited By: Levulett,



amec

Scale
1 inch = 20 feet

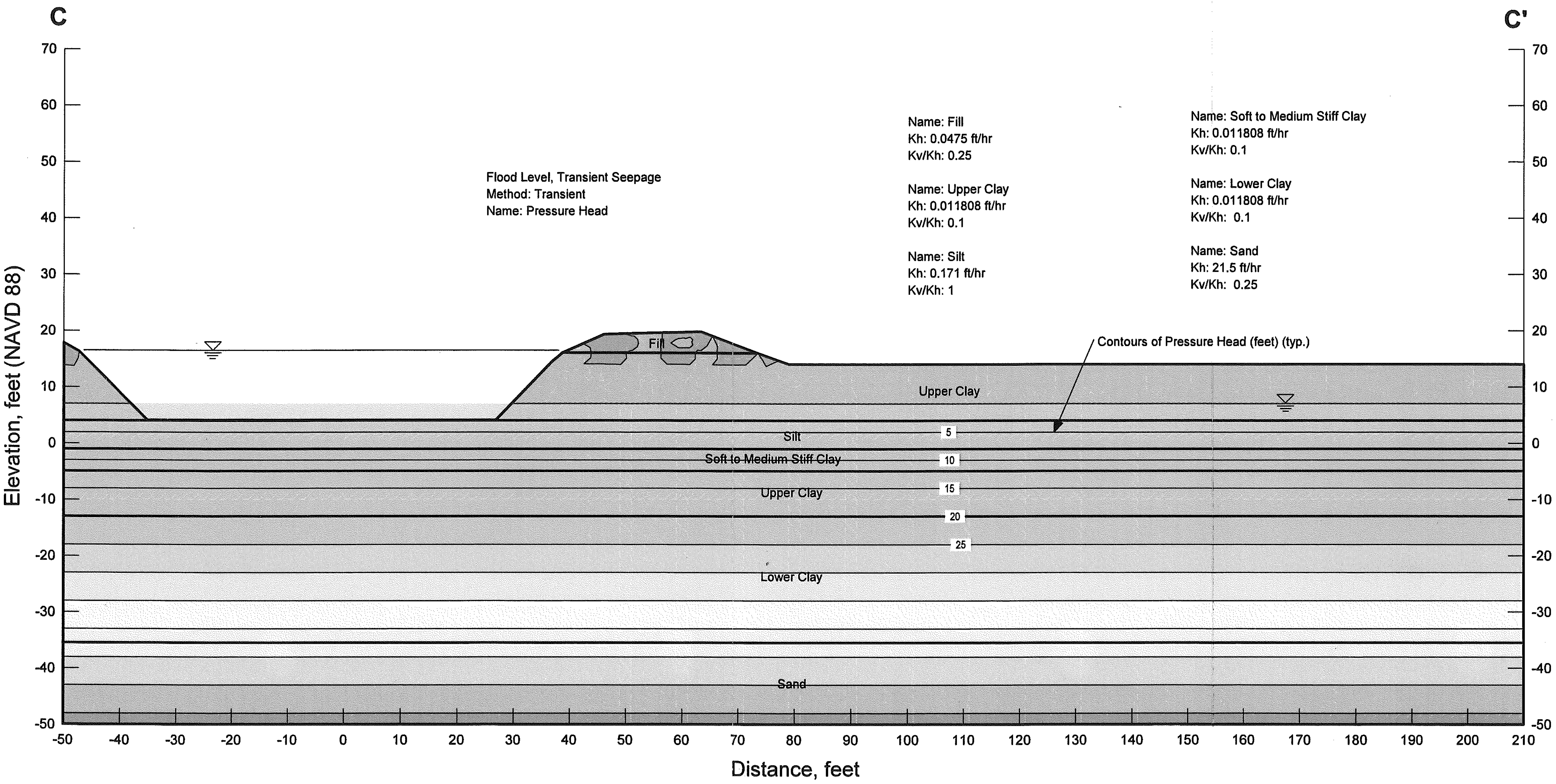
Project Number 13815.003	Drawn D. Levulett
Task 0C.03	Analyzed D. Levulett
Date 6/9/2009	Checked

Lower Penitencia Creek
FEMA Levee Recertification
Milpitas, California

Transient Analysis Result
Elapsed Time: 0 hr
Cross Section C
Levee Station 43+65

Figure
7-1

Directory: I:\Project\13000\13815.003\12000 CALC EVAL ANALYSES\Geo Studio\Section C\No Concrete\C_RS_4440_Transientx10.gsz Date: 6/9/2009 Time: 10:56:22 AM Last Edited By: Levulett



Flood Level, Transient Seepage
Method: Transient
Name: Pressure Head

Name: Fill
Kh: 0.0475 ft/hr
Kv/Kh: 0.25

Name: Upper Clay
Kh: 0.011808 ft/hr
Kv/Kh: 0.1

Name: Silt
Kh: 0.171 ft/hr
Kv/Kh: 1

Name: Soft to Medium Stiff Clay
Kh: 0.011808 ft/hr
Kv/Kh: 0.1

Name: Lower Clay
Kh: 0.011808 ft/hr
Kv/Kh: 0.1

Name: Sand
Kh: 21.5 ft/hr
Kv/Kh: 0.25

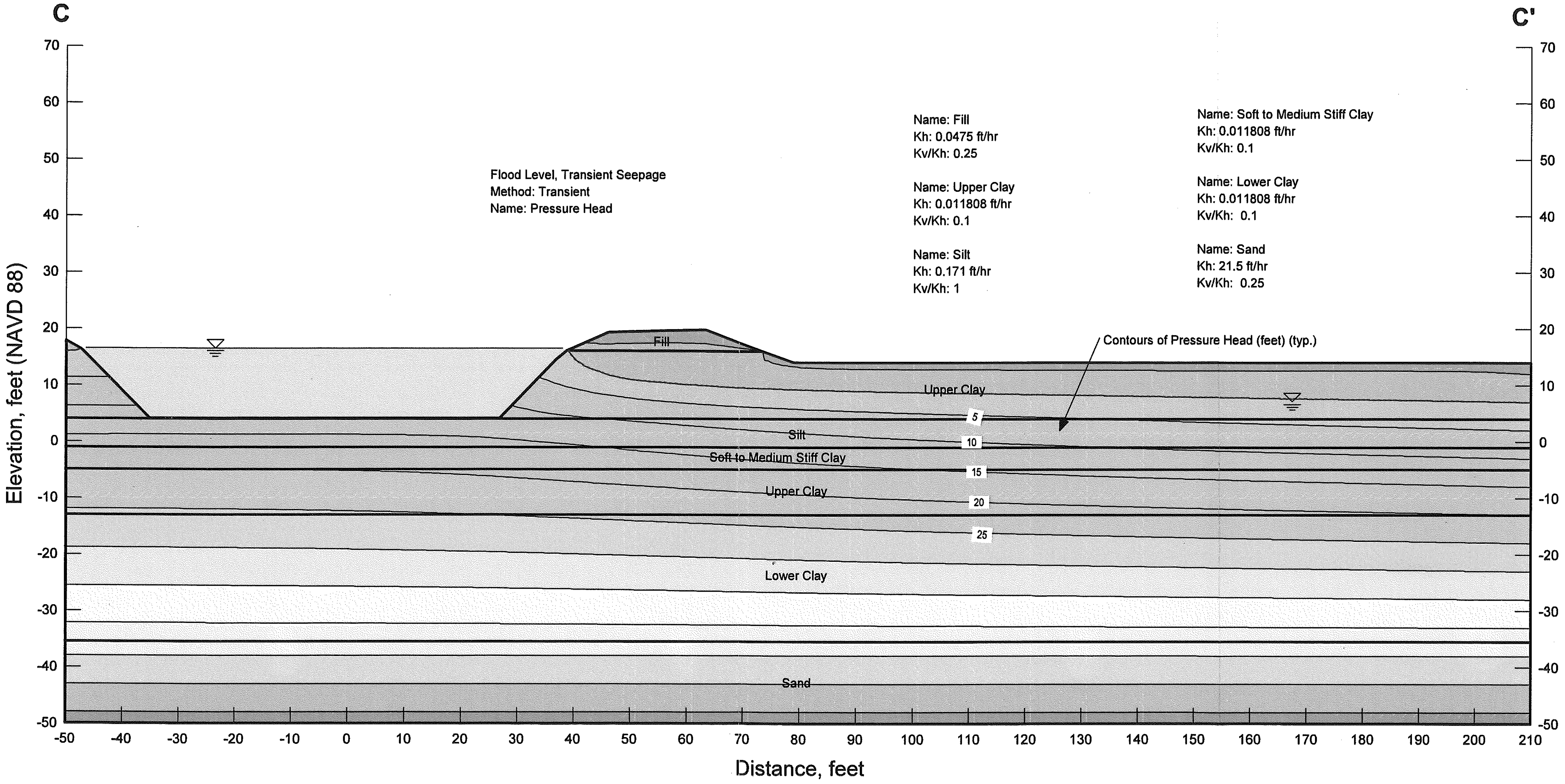
amec

Scale 1 inch = 20 feet	Project Number 13815.003	Drawn D. Levulett	**Lower Penitencia Creek** **FEMA Levee Recertification** **Milpitas, California**	**Transient Analysis Result** Elapsed Time: 0 hr Cross Section C Levee Station 43+65
	Task 0C.03	Analyzed D. Levulett		
	Date 6/9/2009	Checked TJT		

Figure

7-1

Directory: I:\Project\13000\13815.003\12000 CALC EVAL ANALYSES\Geo Studio\Section C\No Concrete\C_RS_4440_Transientx10.gsz Date: 6/9/2009 Time: 10:56:22 AM Last Edited By: Levulett,



amec

Scale
1 inch = 20 feet

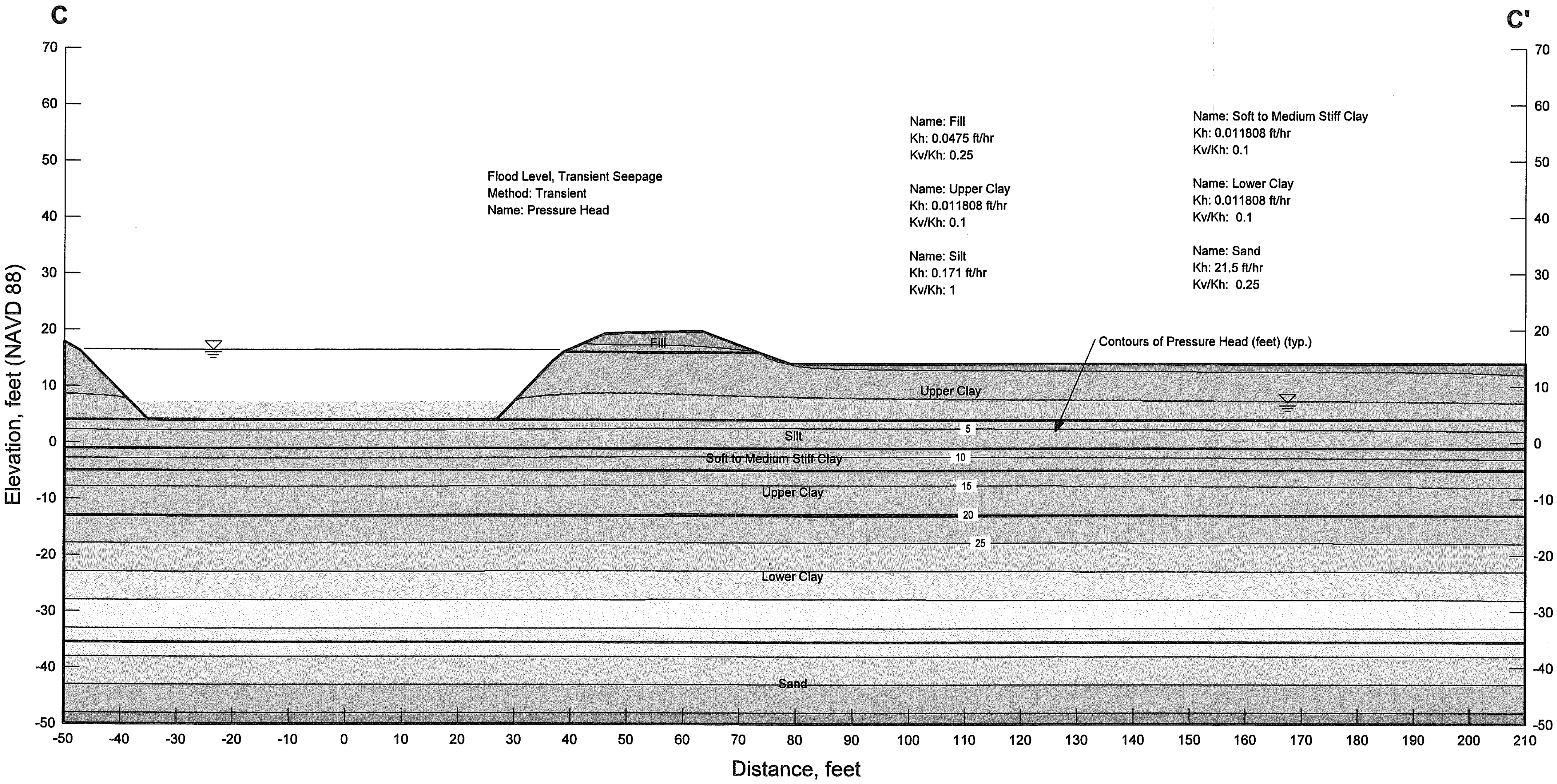
Project Number 13815.003	Drawn D. Levulett
Task 0C.03	Analyzed D. Levulett
Date 6/9/2009	Checked JJJ

Lower Penitencia Creek
FEMA Levee Recertification
Milpitas, California

Transient Analysis Result
Elapsed Time: 24 hr
Cross Section C
Levee Station 43+65

Figure
7-2

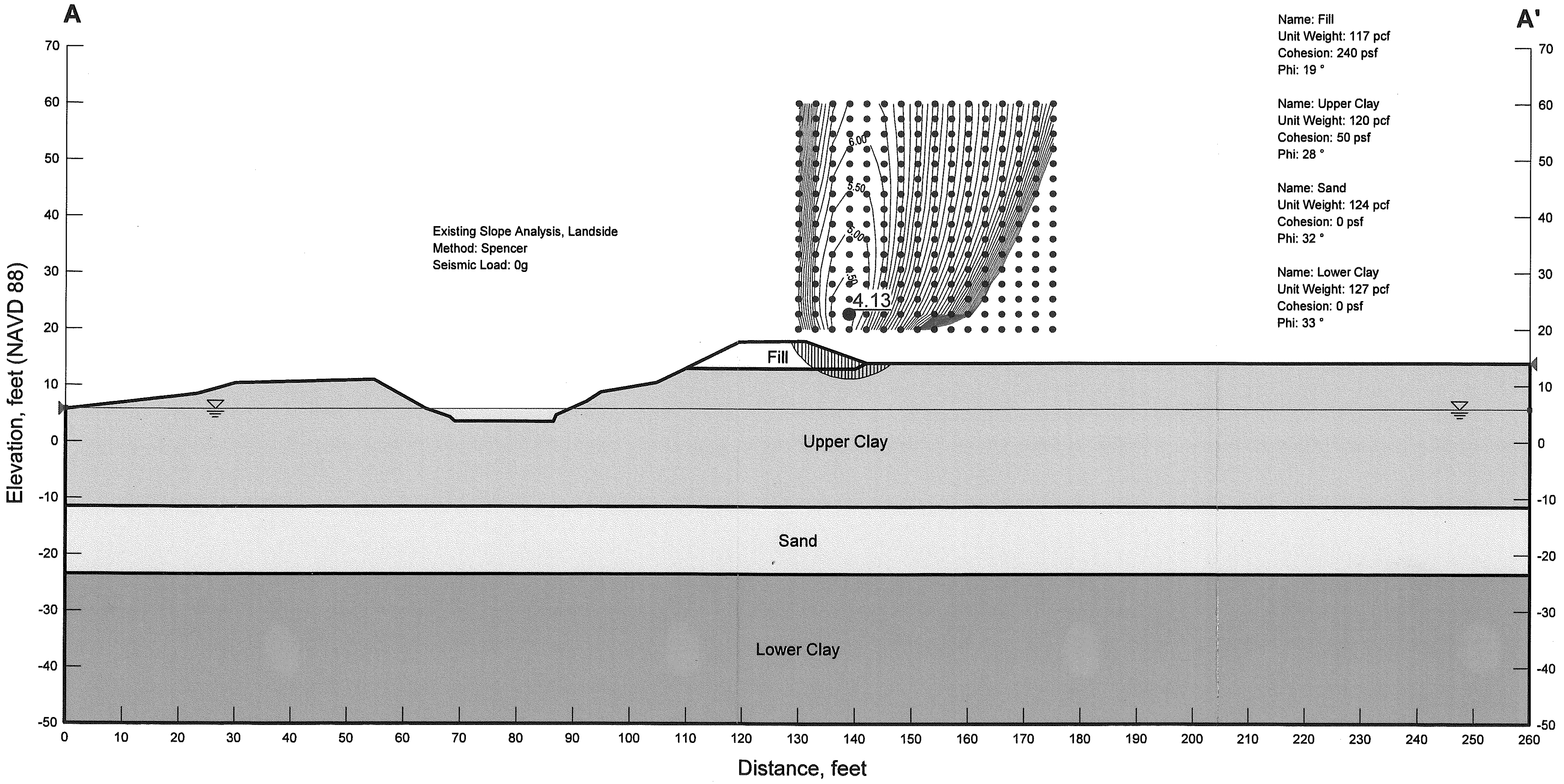
Directory: I:\Project\13000\13815.003\12000 CALC EVAL ANALYSES\Geo Studio\Section C\No Concrete\C_RS_4440_Transientx10.gsz Date: 6/9/2009 Time: 10:56:22 AM Last Edited By: Levulett,



amec

Scale	Project Number 13815.003	Drawn D. Levulett	**Lower Penitencia Creek**	**Transient Analysis Result**	**Figure**
1 inch = 20 feet	Task 0C.03	Analyzed D. Levulett	**FEMA Levee Recertification**	Elapsed Time: 48 hr Cross Section C	7-3
	Date 6/9/2009	Checked JHJ	**Milpitas, California**	Levee Station 43+65	

Directory: I:\Project\13000\13815.003\12000 CALC EVAL ANALYSES\Geo Studio\Section A\A_RS_1640_Existing.gsz Date: 4/24/2009 Time: 10:34:15 AM Last Edited By: Levulett, Darcie



Existing Slope Analysis, Landside
Method: Spencer
Seismic Load: 0g

Name: Fill
Unit Weight: 117 pcf
Cohesion: 240 psf
Phi: 19 °

Name: Upper Clay
Unit Weight: 120 pcf
Cohesion: 50 psf
Phi: 28 °

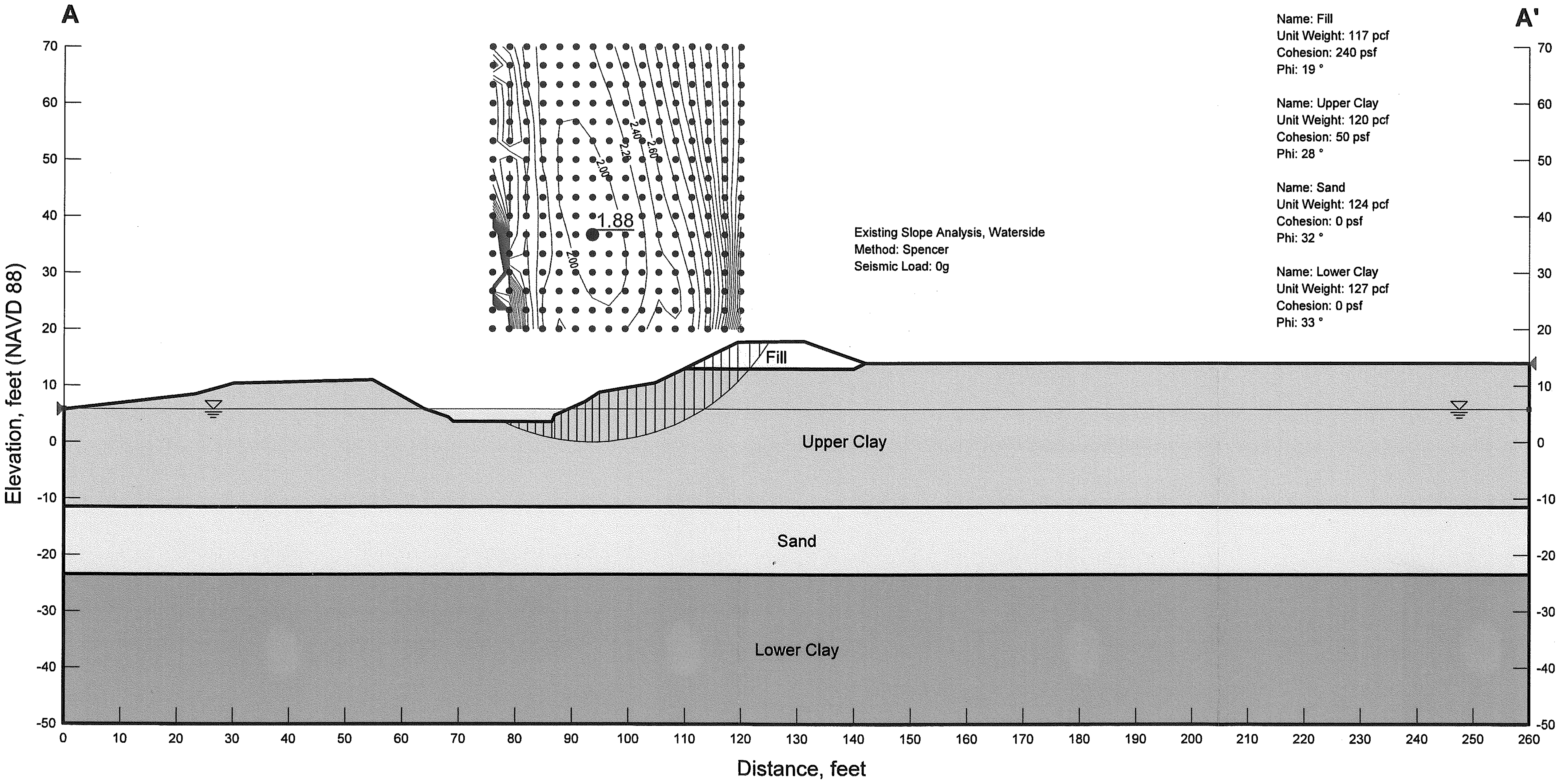
Name: Sand
Unit Weight: 124 pcf
Cohesion: 0 psf
Phi: 32 °

Name: Lower Clay
Unit Weight: 127 pcf
Cohesion: 0 psf
Phi: 33 °

amec

Scale 1 inch = 20 feet	Project Number 13815.003	Drawn D. Levulett	**Lower Penitencia Creek** **FEMA Levee Recertification** **Milpitas, California**	**Existing Slope Stability Analysis** Landside Cross Section A Levee Station 15+65	**Figure** 8-1
	Task 0C.03	Analyzed D. Levulett			
	Date 4/24/2009	Checked PSK			

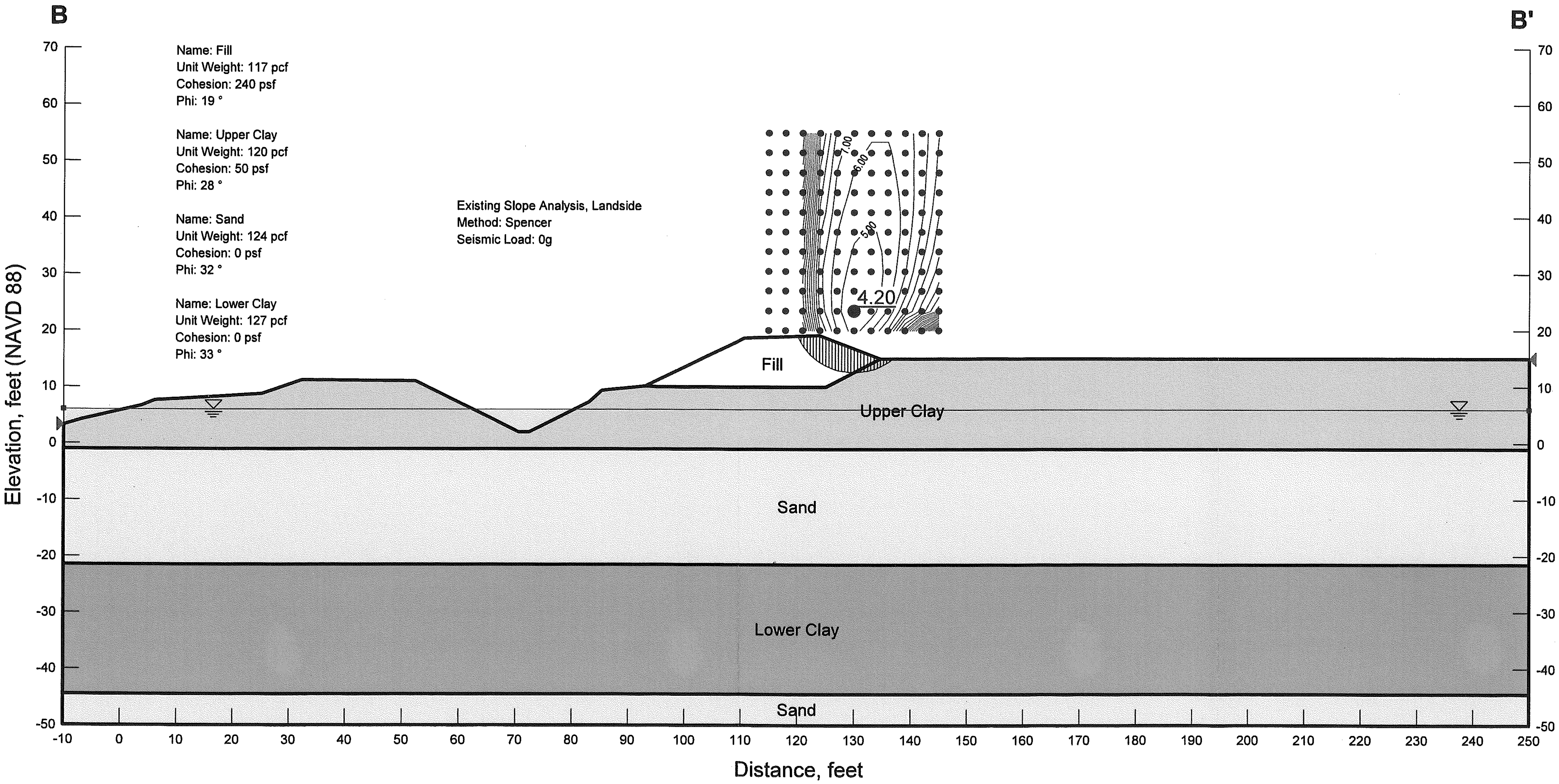
Directory: I:\Project\13000\13815.003\12000 CALC EVAL ANALYSES\Geo Studio\Section AA_RS_1640_Existing.gsz Date: 4/24/2009 Time: 10:34:15 AM Last Edited By: Levulett, Darcie



amec

Scale 1 inch = 20 feet	Project Number 13815.003	Drawn D. Levulett	**Lower Penitencia Creek** **FEMA Levee Recertification** **Milpitas, California**	**Existing Slope Stability Analysis** Waterside Cross Section A Levee Station 15+65	**Figure** 8-2
	Task 0C.03	Analyzed D. Levulett			
	Date 4/24/2009	Checked PSK			

Directory: I:\Project\13000\13815.003\12000 CALC EVAL ANALYSES\Geo Studio\Section BIB_RS_3026_Existing.gsz Date: 5/21/2009 Time: 9:55:42 AM Last Edited By: Levulett, Darcie



Name: Fill
Unit Weight: 117 pcf
Cohesion: 240 psf
Phi: 19 °

Name: Upper Clay
Unit Weight: 120 pcf
Cohesion: 50 psf
Phi: 28 °

Name: Sand
Unit Weight: 124 pcf
Cohesion: 0 psf
Phi: 32 °

Name: Lower Clay
Unit Weight: 127 pcf
Cohesion: 0 psf
Phi: 33 °

Existing Slope Analysis, Landside
Method: Spencer
Seismic Load: 0g

amec

Scale
1 inch = 20 feet

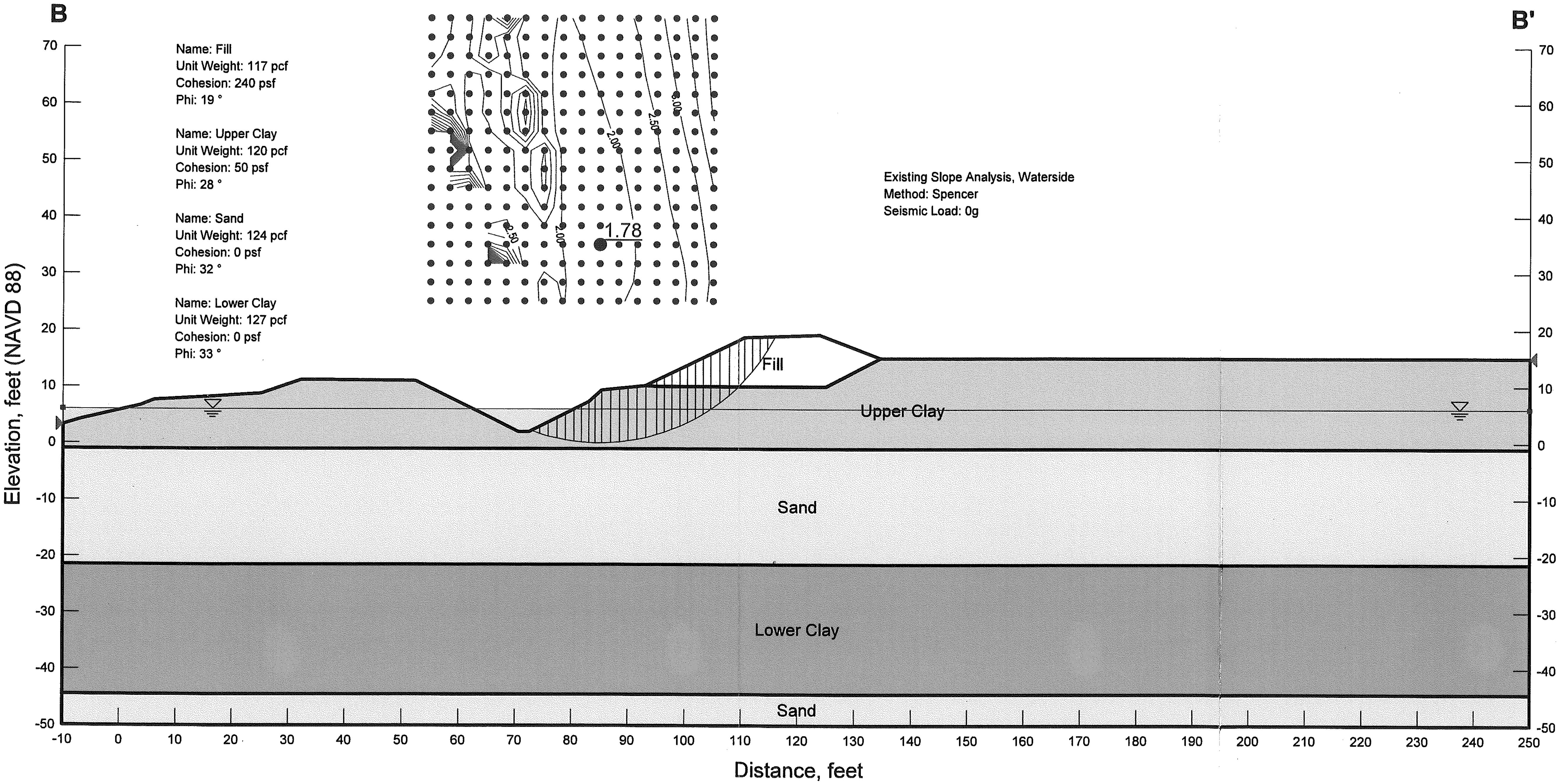
Project Number 13815.003	Drawn D. Levulett
Task 0C.03	Analyzed D. Levulett
Date 5/21/2009	Checked psk

Lower Penitencia Creek
FEMA Levee Recertification
Milpitas, California

Existing Slope Stability Analysis
Landside
Cross Section B
Levee Station 26+51

Figure
8-3

Directory: I:\Project\13000\13815.003\12000 CALC EVAL ANALYSES\Geo Studio\Section BIB_RS_3026_Existing.gsz Date: 5/21/2009 Time: 9:58:32 AM Last Edited By: Levulett, Darcie



amec

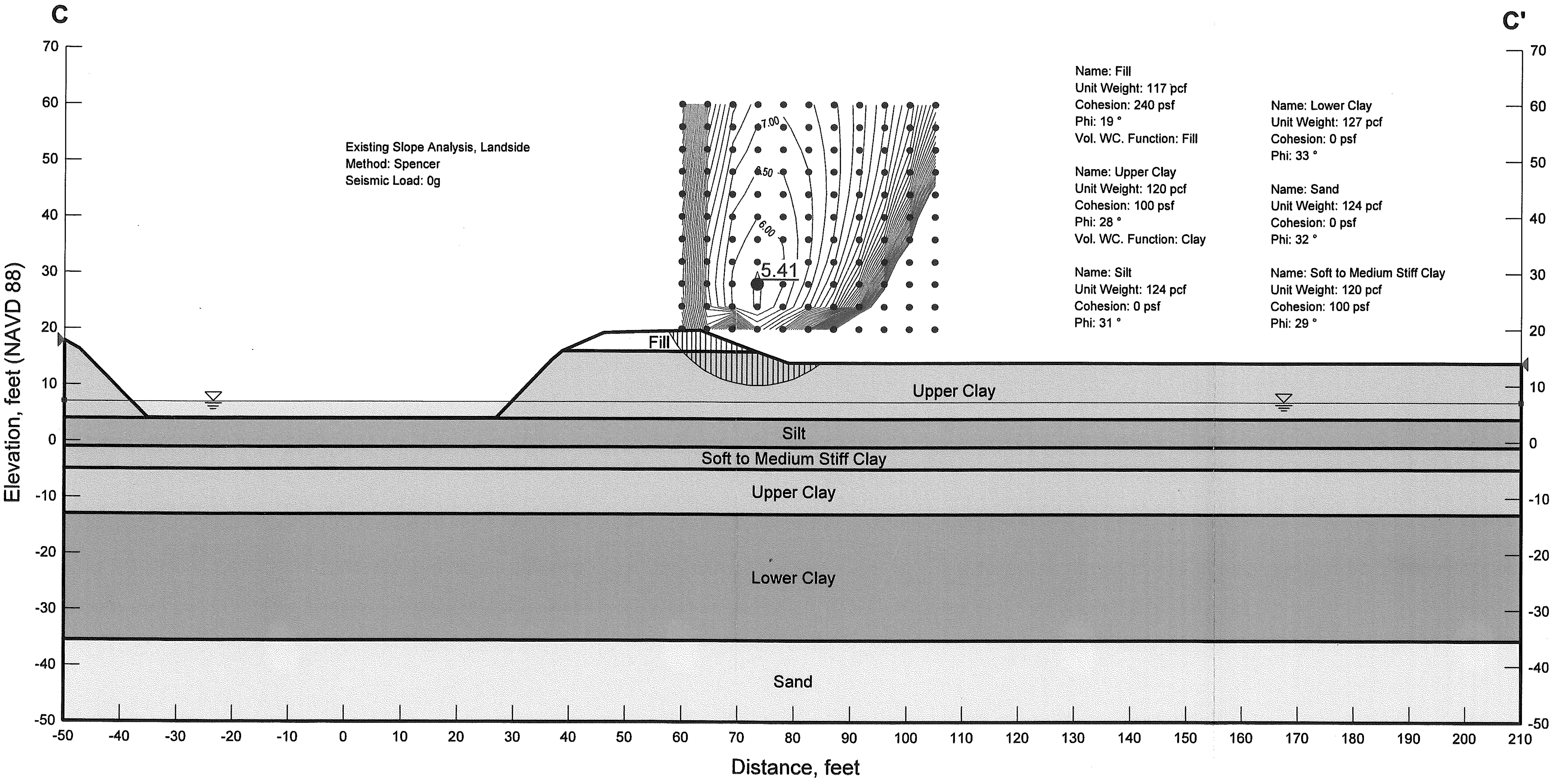
Scale 1 inch = 20 feet	Project Number 13815.003	Drawn D. Levulett
	Task 0C.03	Analyzed D. Levulett
	Date 5/21/2009	Checked PSK

Lower Penitencia Creek
FEMA Levee Recertification
Milpitas, California

Existing Slope Stability Analysis
Waterside
Cross Section B
Levee Station 26+51

Figure
8-4

Directory: I:\Project\13000\13815.003\12000 CALC EVAL ANALYSES\Geo Studio\Section C\No Concrete\C_RS_4440_Existing_US2.gsz Date: 6/2/2009 Time: 12:17:51 PM Last Edited By: Levulett



Existing Slope Analysis, Landside
Method: Spencer
Seismic Load: 0g

Name: Fill
Unit Weight: 117 pcf
Cohesion: 240 psf
Phi: 19 °
Vol. WC. Function: Fill

Name: Upper Clay
Unit Weight: 120 pcf
Cohesion: 100 psf
Phi: 28 °
Vol. WC. Function: Clay

Name: Silt
Unit Weight: 124 pcf
Cohesion: 0 psf
Phi: 31 °

Name: Lower Clay
Unit Weight: 127 pcf
Cohesion: 0 psf
Phi: 33 °

Name: Sand
Unit Weight: 124 pcf
Cohesion: 0 psf
Phi: 32 °

Name: Soft to Medium Stiff Clay
Unit Weight: 120 pcf
Cohesion: 100 psf
Phi: 29 °

amec

Scale
1 inch = 20 feet

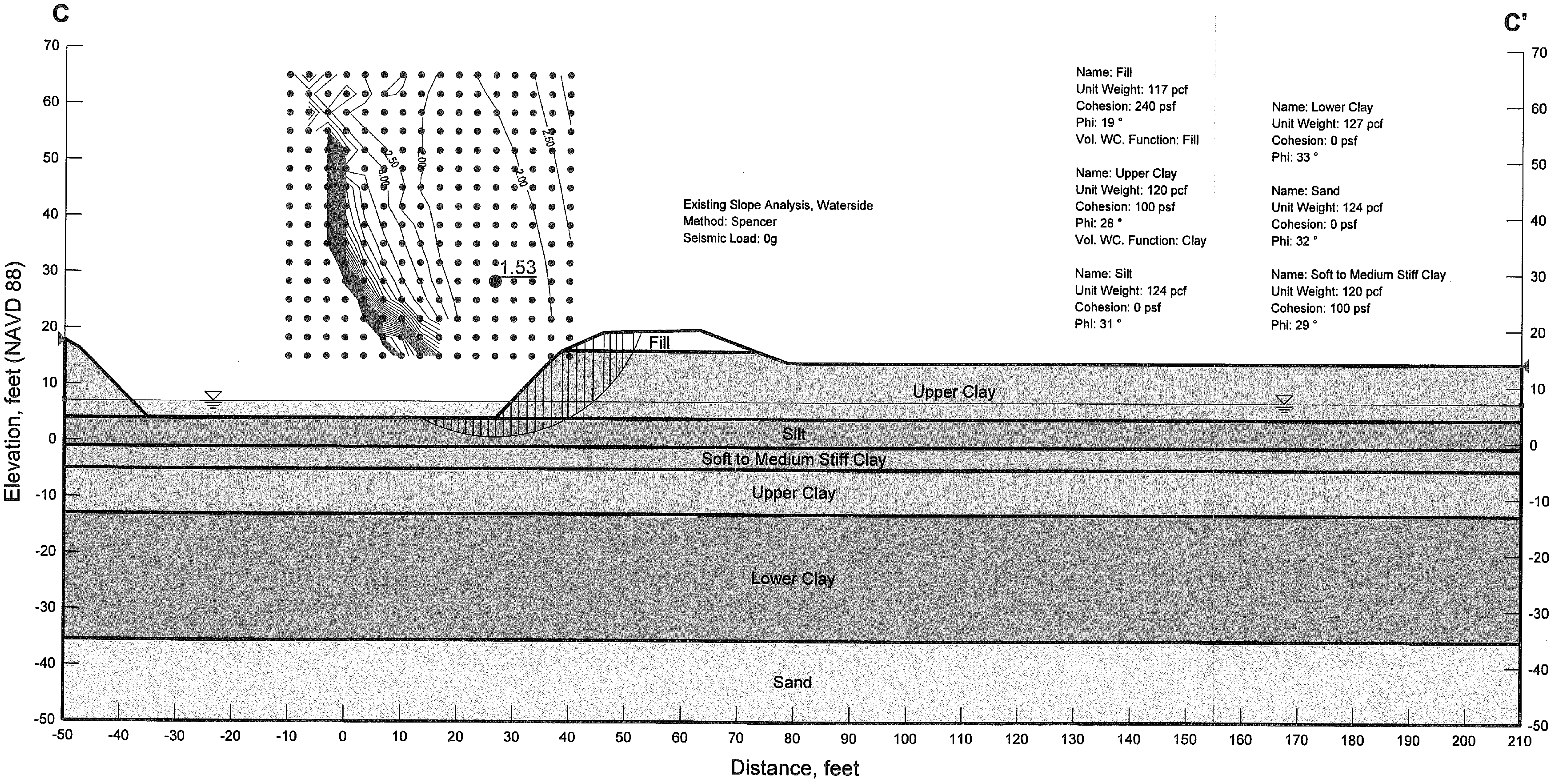
Project Number 13815.003	Drawn D. Levulett
Task 0C.03	Analyzed D. Levulett
Date 6/2/2009	Checked psk

Lower Penitencia Creek
FEMA Levee Recertification
Milpitas, California

Existing Slope Stability Analysis
Landside
Cross Section C
Levee Station 43+65

Figure
8-5

Directory: I:\Project\13000\13815.003\12000 CALC EVAL ANALYSES\Geo Studio\Section C\No Concrete\C_RS_4440_Existing_US2.gsz Date: 6/2/2009 Time: 12:17:51 PM Last Edited By: Levulett



Existing Slope Analysis, Waterside
Method: Spencer
Seismic Load: 0g

Name: Fill
Unit Weight: 117 pcf
Cohesion: 240 psf
Phi: 19 °
Vol. WC. Function: Fill

Name: Upper Clay
Unit Weight: 120 pcf
Cohesion: 100 psf
Phi: 28 °
Vol. WC. Function: Clay

Name: Silt
Unit Weight: 124 pcf
Cohesion: 0 psf
Phi: 31 °

Name: Lower Clay
Unit Weight: 127 pcf
Cohesion: 0 psf
Phi: 33 °

Name: Sand
Unit Weight: 124 pcf
Cohesion: 0 psf
Phi: 32 °

Name: Soft to Medium Stiff Clay
Unit Weight: 120 pcf
Cohesion: 100 psf
Phi: 29 °

amec

Scale
1 inch = 20 feet

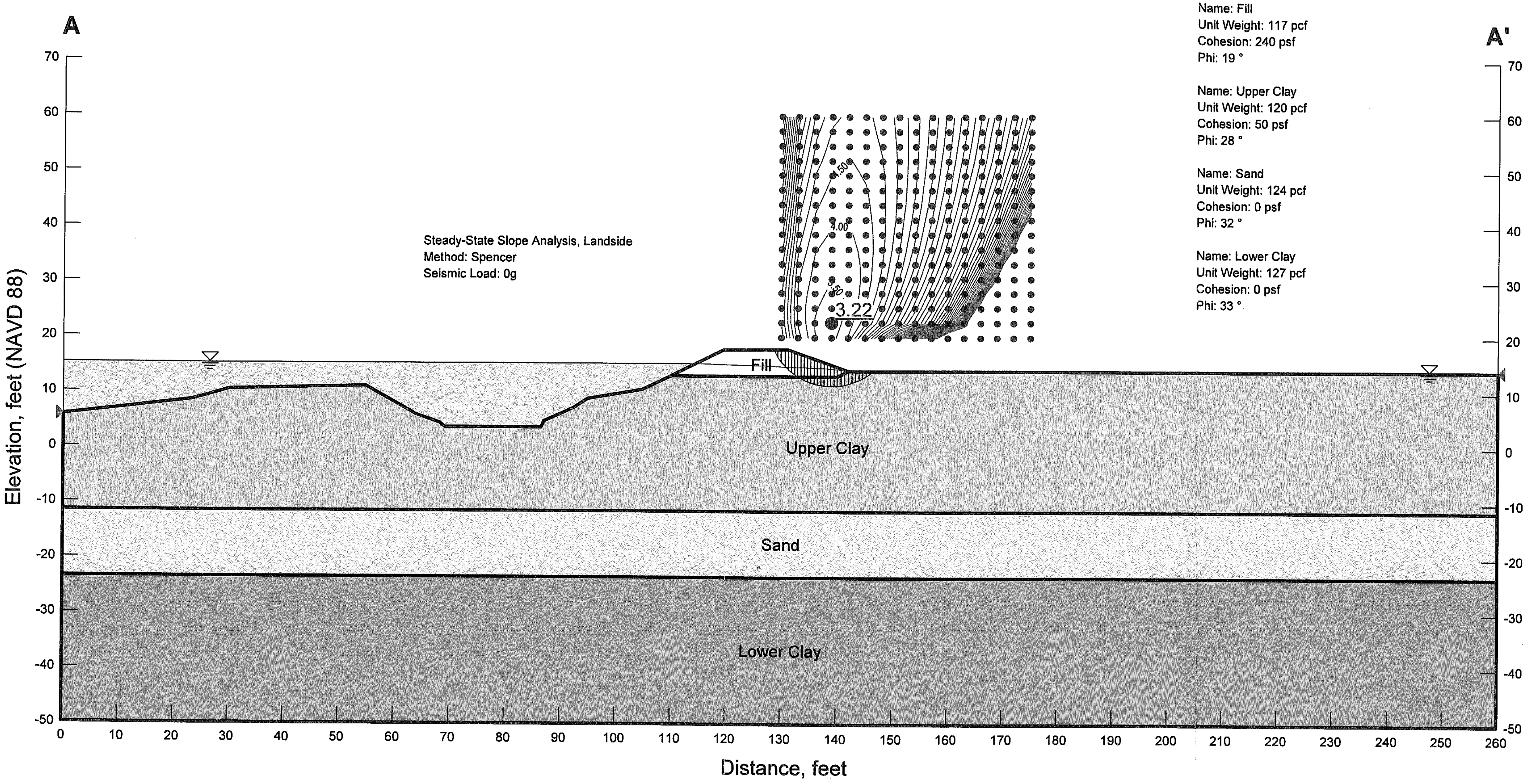
Project Number 13815.003	Drawn D. Levulett
Task 0C.03	Analyzed D. Levulett
Date 6/2/2009	Checked psk

Lower Penitencia Creek
FEMA Levee Recertification
Milpitas, California

Existing Slope Stability Analysis
Waterside
Cross Section C
Levee Station 43+65

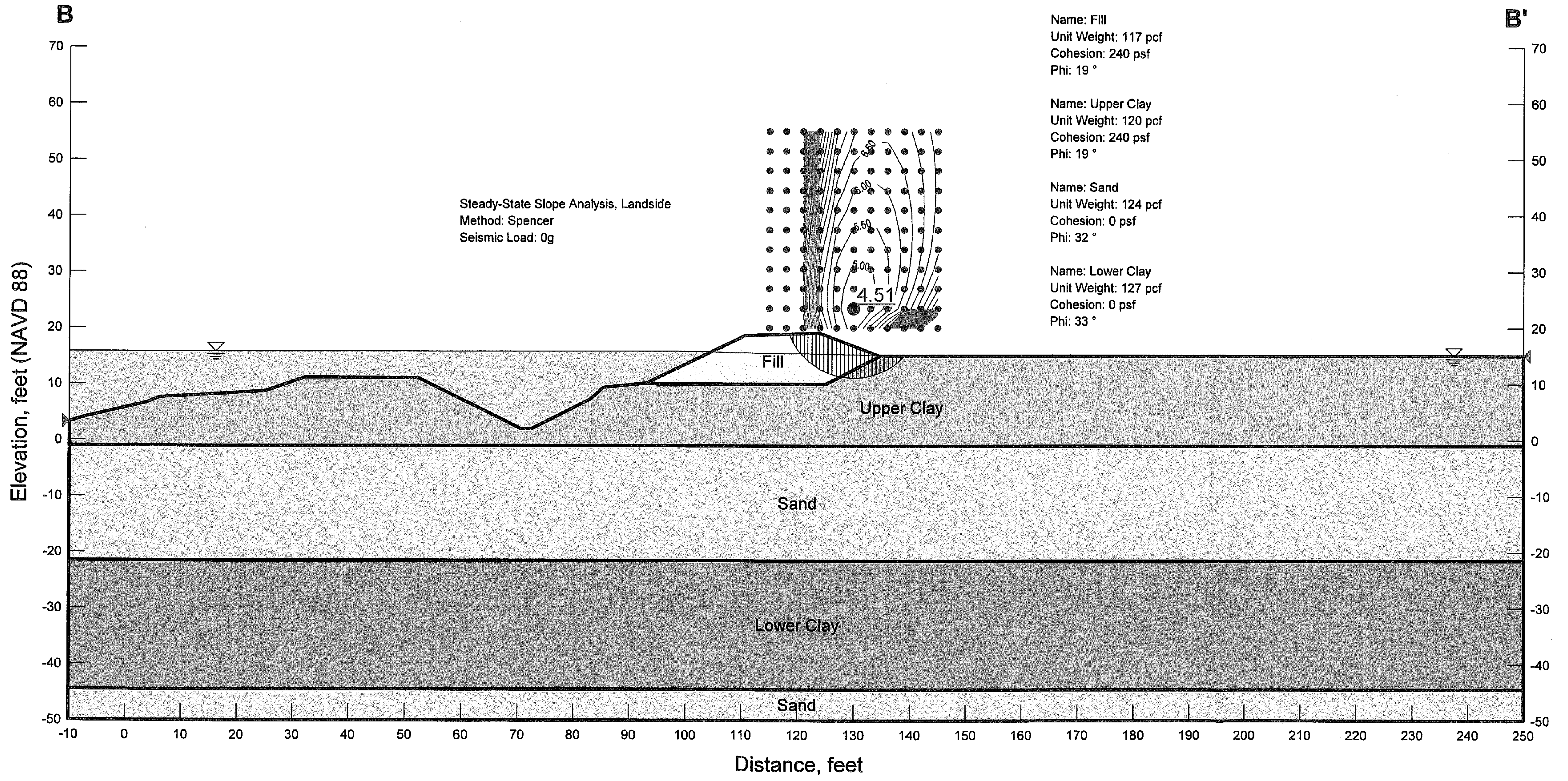
Figure
8-6

Directory: I:\Project\13000\13815.003\12000 CALC EVAL ANALYSES\Geo Studio\Section AV_RS_1640_SteadyState.gsz Date: 6/15/2009 Time: 10:25:49 AM Last Edited By: Levulett, Darcie



amec	**Scale** 1 inch = 20 feet	Project Number 13815.003	Drawn D. Levulett	**Lower Penitencia Creek** **FEMA Levee Recertification** **Milpitas, California**	**Steady-State Slope Stability Result** Landside Cross Section A Levee Station 15+65
		Task 0C.03	Analyzed D. Levulett		**Figure** 9-1
		Date 6/15/2009	Checked		

Directory: I:\Project\13000\13815.003\12000 CALC EVAL ANALYSES\Geo Studio\Section B\B_RS_3026_SteadyState.gsz Date: 6/15/2009 Time: 11:11:19 AM Last Edited By: Levulett, Darcie



amec

Scale
1 inch = 20 feet

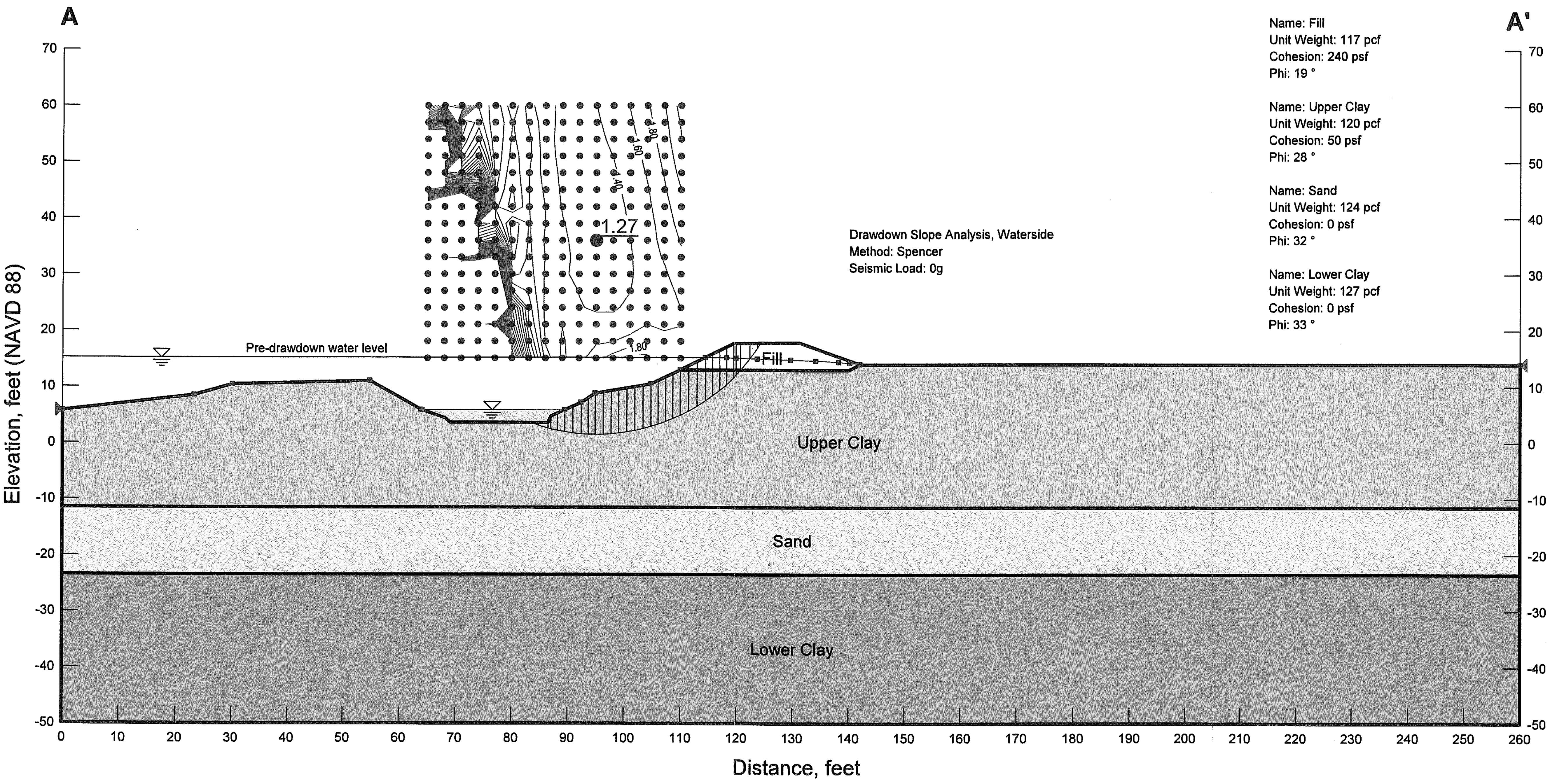
Project Number 13815.003	Drawn D. Levulett
Task 0C.03	Analyzed D. Levulett
Date 6/15/2009	Checked

Lower Penitencia Creek
FEMA Levee Recertification
Milpitas, California

Steady-State Slope Stability Result
Landside
Cross Section B
Levee Station 26+51

Figure
9-2

Directory: I:\Project\13000\13815.003\12000 CALC EVAL ANALYSES\Geo Studio\Section A\A_RS_1640_Drawdown.gsz Date: 6/15/2009 Time: 10:47:53 AM Last Edited By: Levulett, Darcie



amec

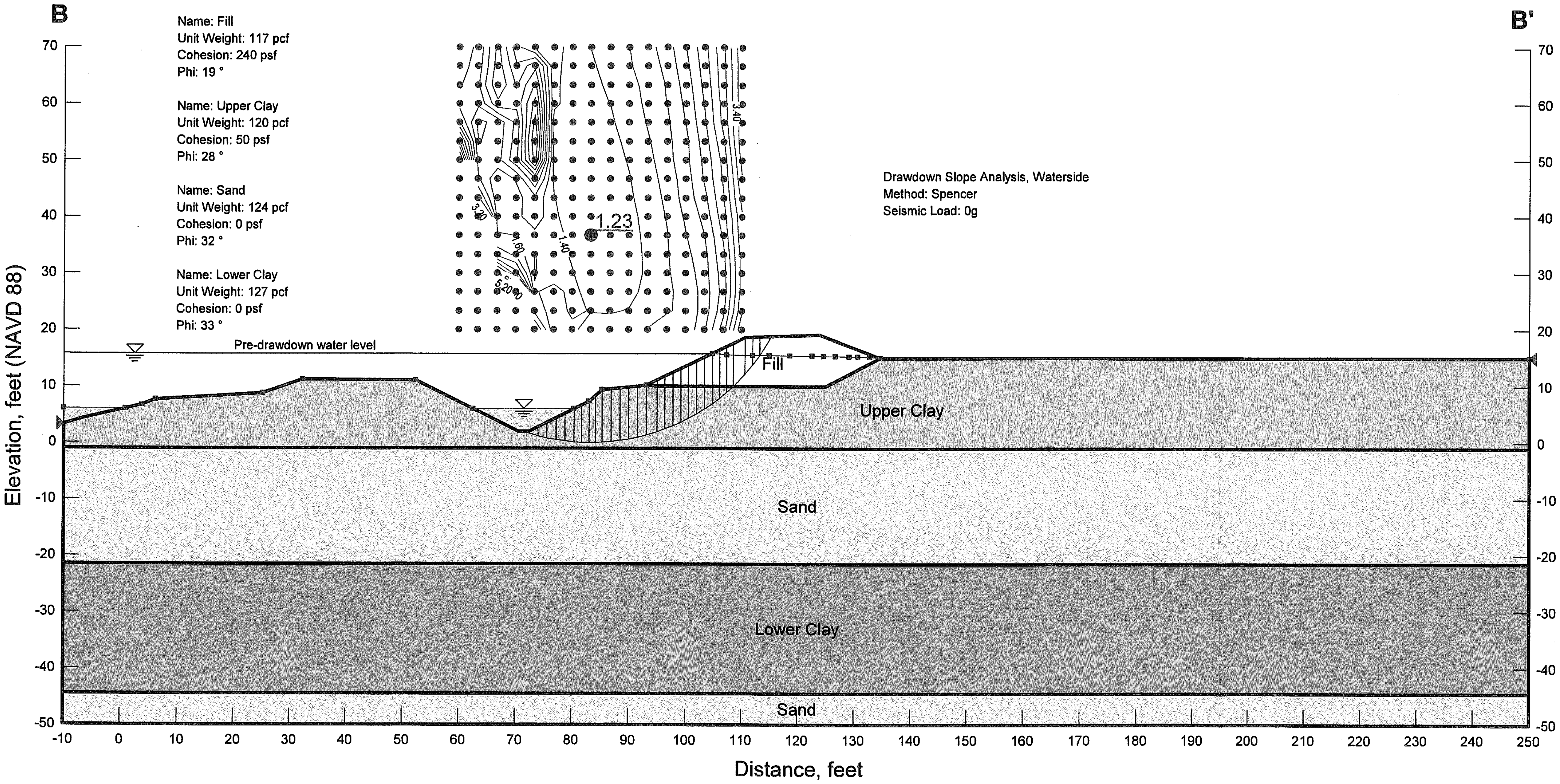
Scale	Project Number 13815.003	Drawn D. Levulett
1 inch = 20 feet	Task 0C.03	Analyzed D. Levulett
	Date 6/15/2009	Checked

Lower Penitencia Creek
FEMA Levee Recertification
Milpitas, California

Drawdown Slope Stability Result
Waterside
Cross Section A
Levee Station 15+65

Figure
10-1

Directory: I:\Project\13000\13815.003\12000 CALC EVAL ANALYSES\Geo Studio\Section B\B_RS_3026_Drawdown.gsz Date: 6/15/2009 Time: 11:16:14 AM Last Edited By: Levulett, Darcie



amec

Scale
1 inch = 20 feet

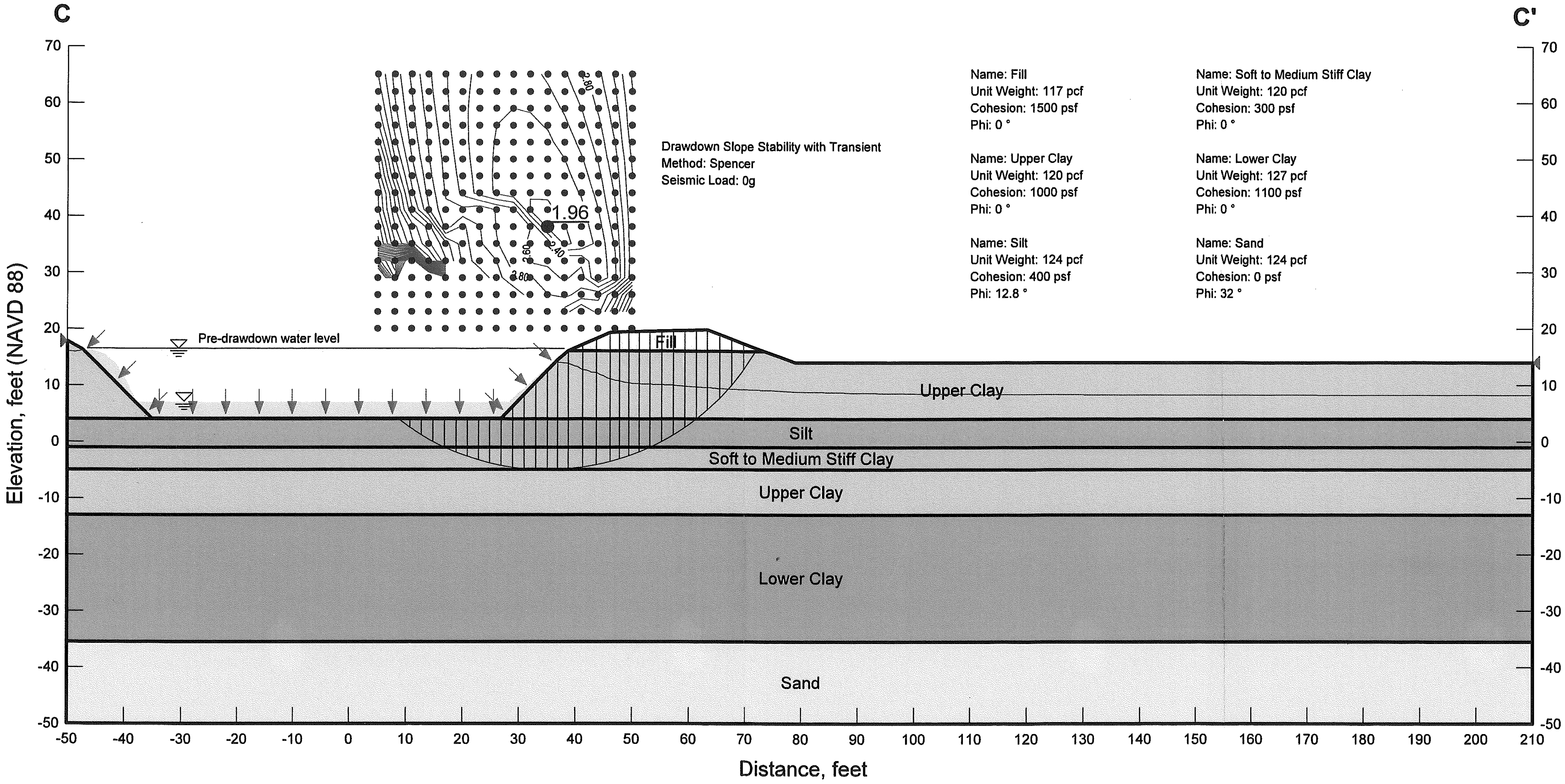
Project Number 13815.003	Drawn D. Levulett
Task 0C.03	Analyzed D. Levulett
Date 6/15/2009	Checked

Lower Penitencia Creek
FEMA Levee Recertification
Milpitas, California

Drawdown Slope Stability Result
Waterside
Cross Section B
Levee Station 26+51

Figure
10-2

Directory: I:\Project\13000\13815.003\12000 CALC EVAL ANALYSES\Geo Studio\Section C\No Concrete\C_RS_4440_Transientx10_DD.gsz Date: 6/9/2009 Time: 11:18:47 AM Last Edited By: Lev



amec

Scale
1 inch = 20 feet

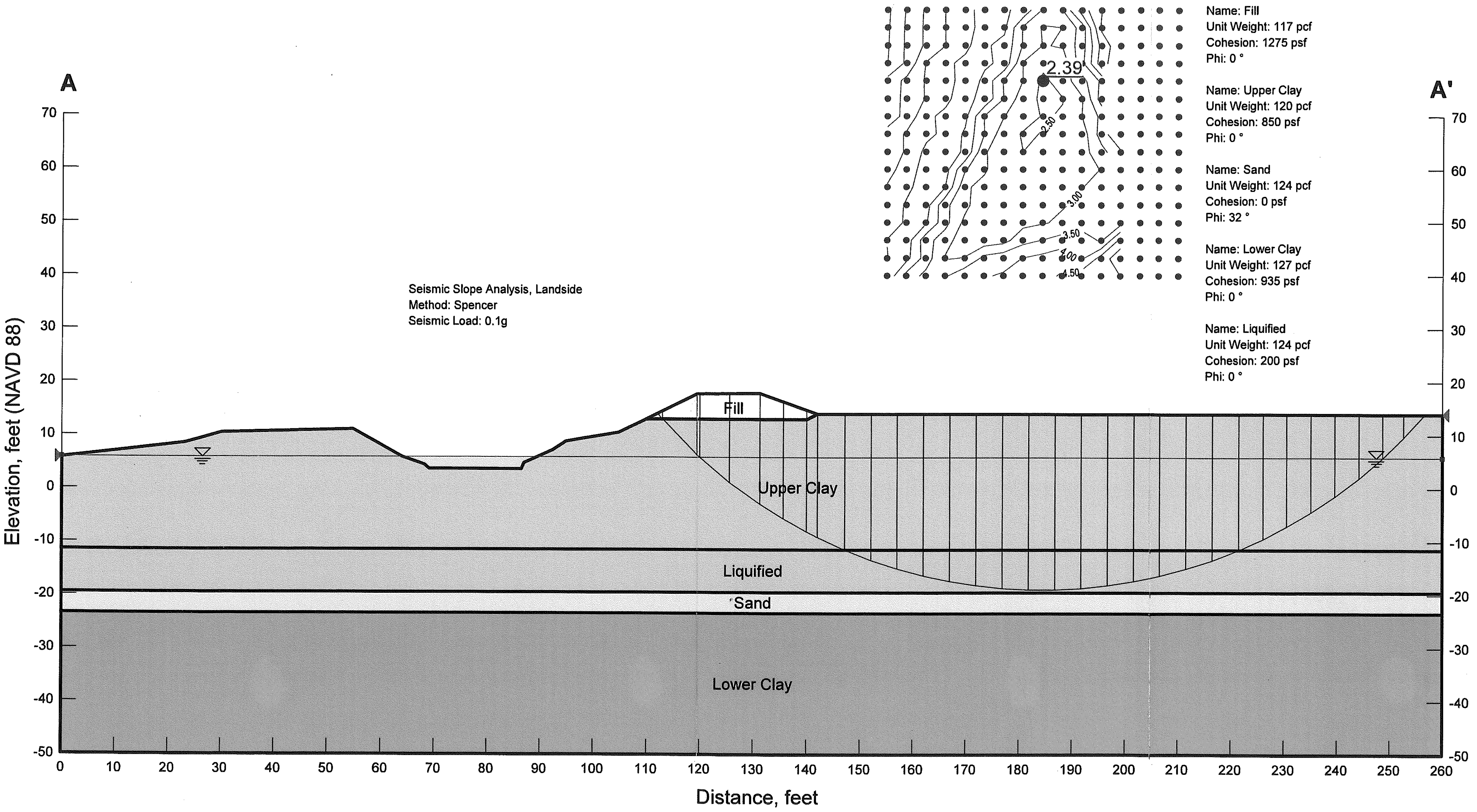
Project Number 13815.003	Drawn D. Levulett
Task 0C.03	Analyzed D. Levulett
Date 6/9/2009	Checked

Lower Penitencia Creek
FEMA Levee Recertification
Milpitas, California

Drawdown Slope Stability Result
Waterside
Cross Section C
Levee Station 43+65

Figure
10-3

Directory: I:\Project\13000\13815.003\12000 CALC EVAL ANALYSES\Geo Studio\Section AA_RS_1640_Seismic.gsz Date: 7/8/2009 Time: 9:07:17 AM Last Edited By: Levulett, Darcie



Seismic Slope Analysis, Landside
Method: Spencer
Seismic Load: 0.1g

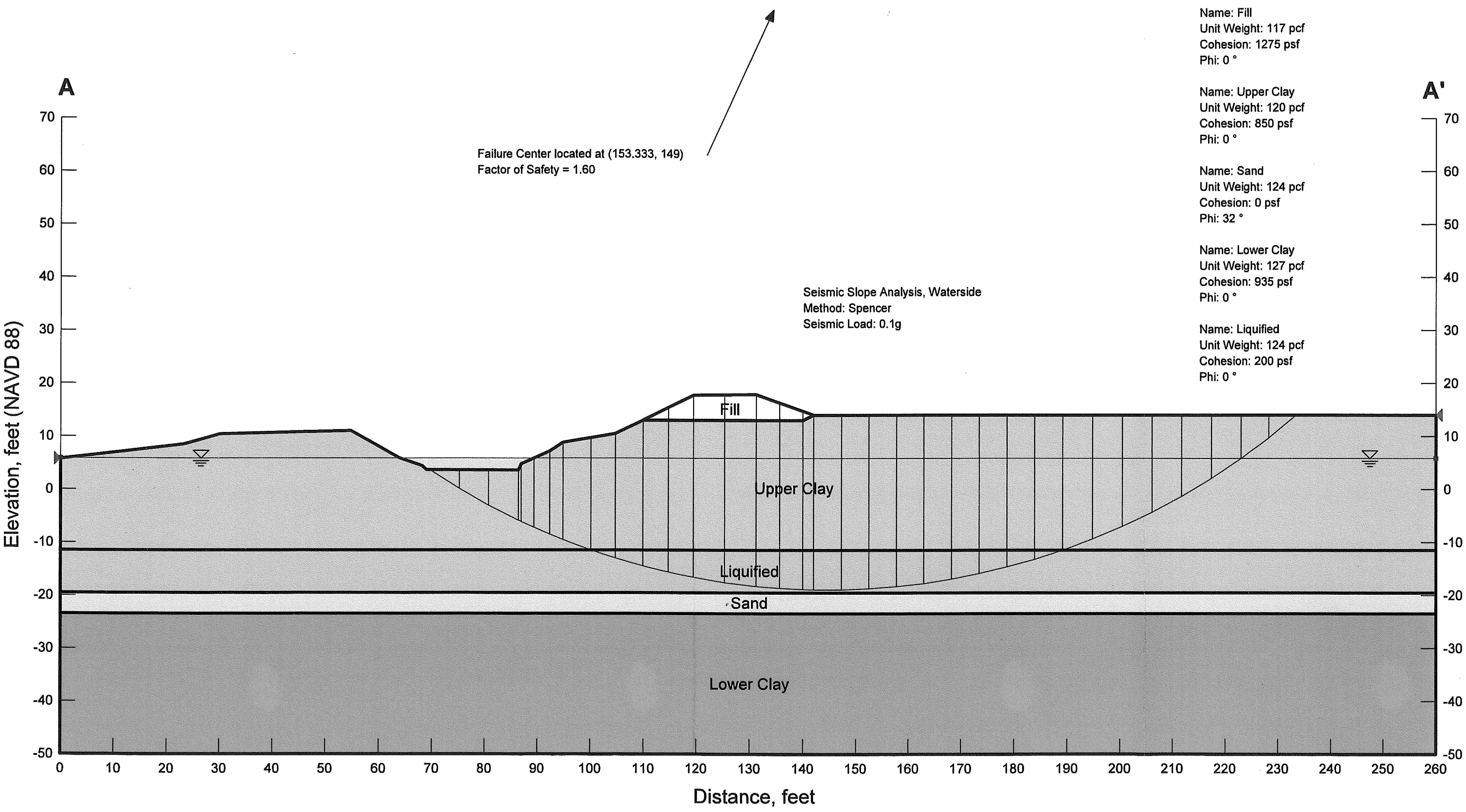
amec

Scale 1 inch = 20 feet	Project Number 13815.003	Drawn D. Levulett	**Lower Penitencia Creek** **FEMA Levee Recertification** **Milpitas, California**	**Seismic Screening Analysis** Landside Cross Section A Levee Station 15+65
	Task 0C.03	Analyzed D. Levulett		
	Date 7/8/2009	Checked		

Figure

11-1

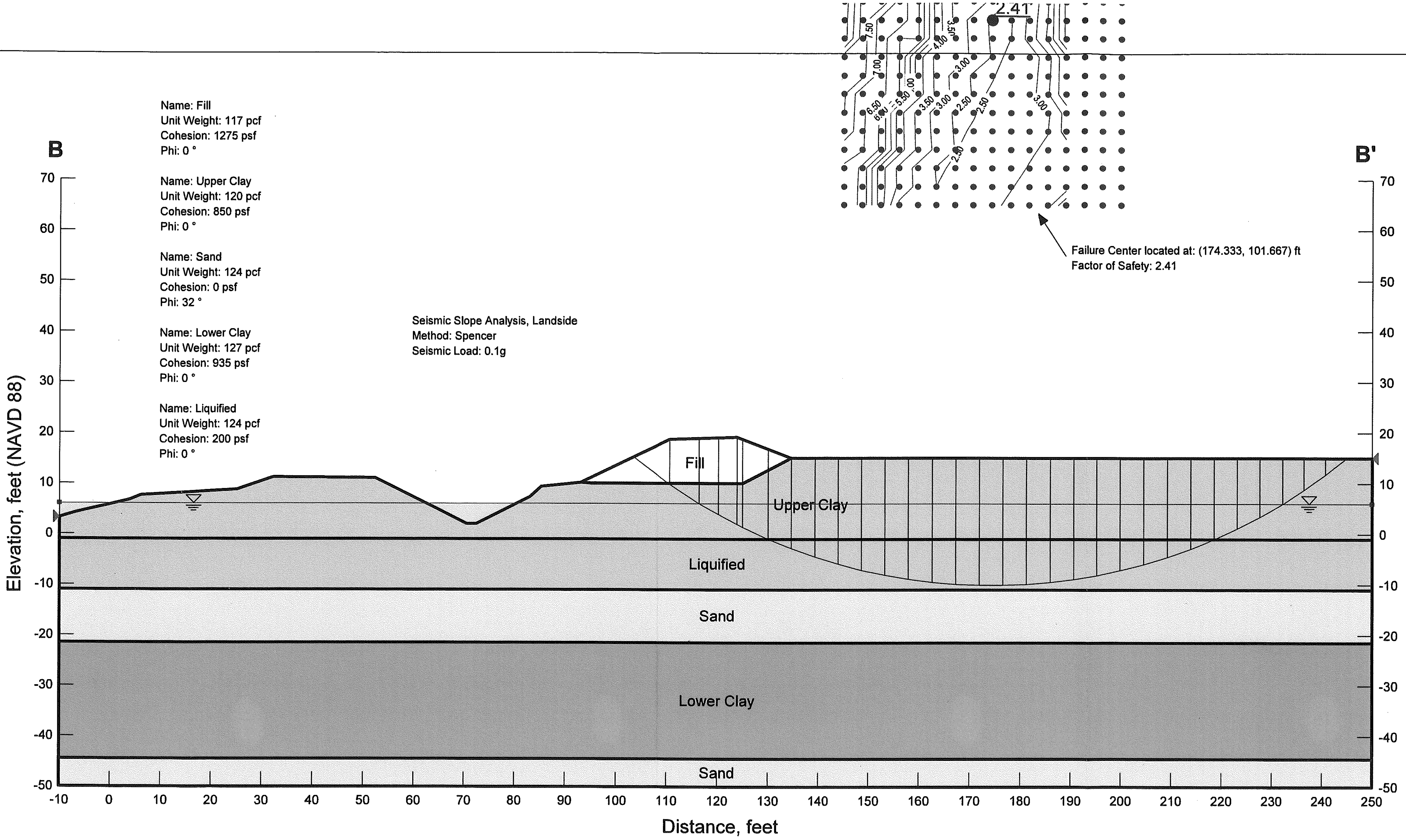
Directory: I:\Project\13000\13815.003\12000 CALC EVAL ANALYSES\Geo Studio\Section A\A_RS_1640_Seismic.gsz Date: 7/8/2009 Time: 9:07:17 AM Last Edited By: Levulett, Darcie



amec

Scale 1 inch = 20 feet	Project Number 13815.003	Drawn D. Levulett	**Lower Penitencia Creek** **FEMA Levee Recertification** **Milpitas, California**	**Seismic Screening Analysis** Waterside Cross Section A Levee Station 15+65	**Figure** 11-2
	Task 0C.03	Analyzed D. Levulett			
	Date 7/8/2009	Checked			

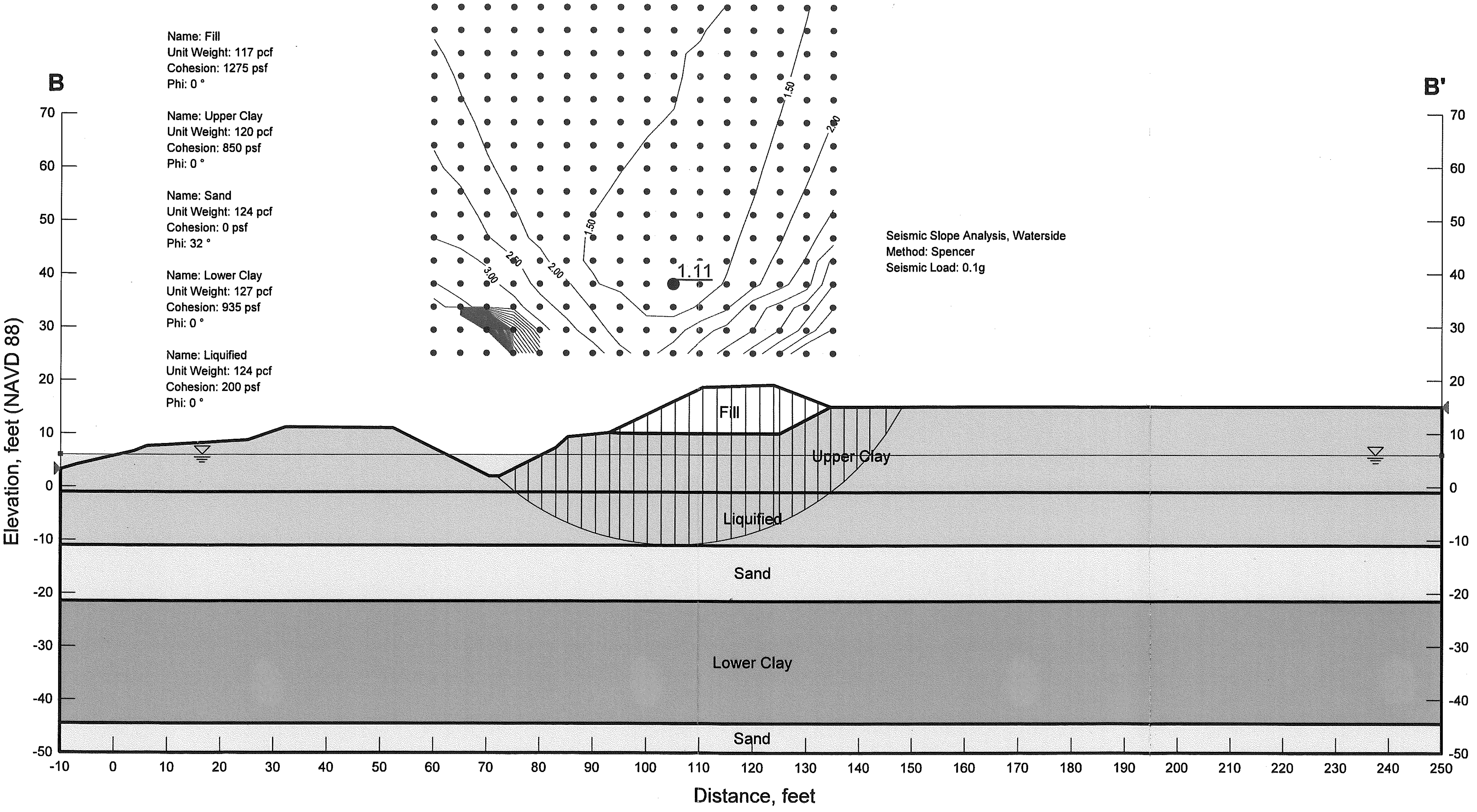
Directory: I:\Project\13000\13815.003\12000 CALC EVAL ANALYSES\Geo Studio\Section B\B_RS_3026_Seismic.gsz Date: 7/8/2009 Time: 9:10:53 AM Last Edited By: Levulett, Darcie



amec

Scale	Project Number 13815.003	Drawn D. Levulett	Lower Penitencia Creek FEMA Levee Recertification Milpitas, California	Seismic Screening Analysis Landside Cross Section B Levee Station 26+51	Figure 11-3
1 inch = 20 feet	Task 0C.03	Analyzed D. Levulett			
	Date 7/8/2009	Checked JJ			

Directory: I:\Project\13000\13815.003\12000 CALC EVAL ANALYSES\Geo Studio\Section B\B_RS_3026_Seismic.gsz Date: 7/8/2009 Time: 9:10:53 AM Last Edited By: Levulett, Darcie



amec

Scale
1 inch = 20 feet

Project Number
13815.003

Task
0C.03

Date
7/8/2009

Drawn
D. Levulett

Analyzed
D. Levulett

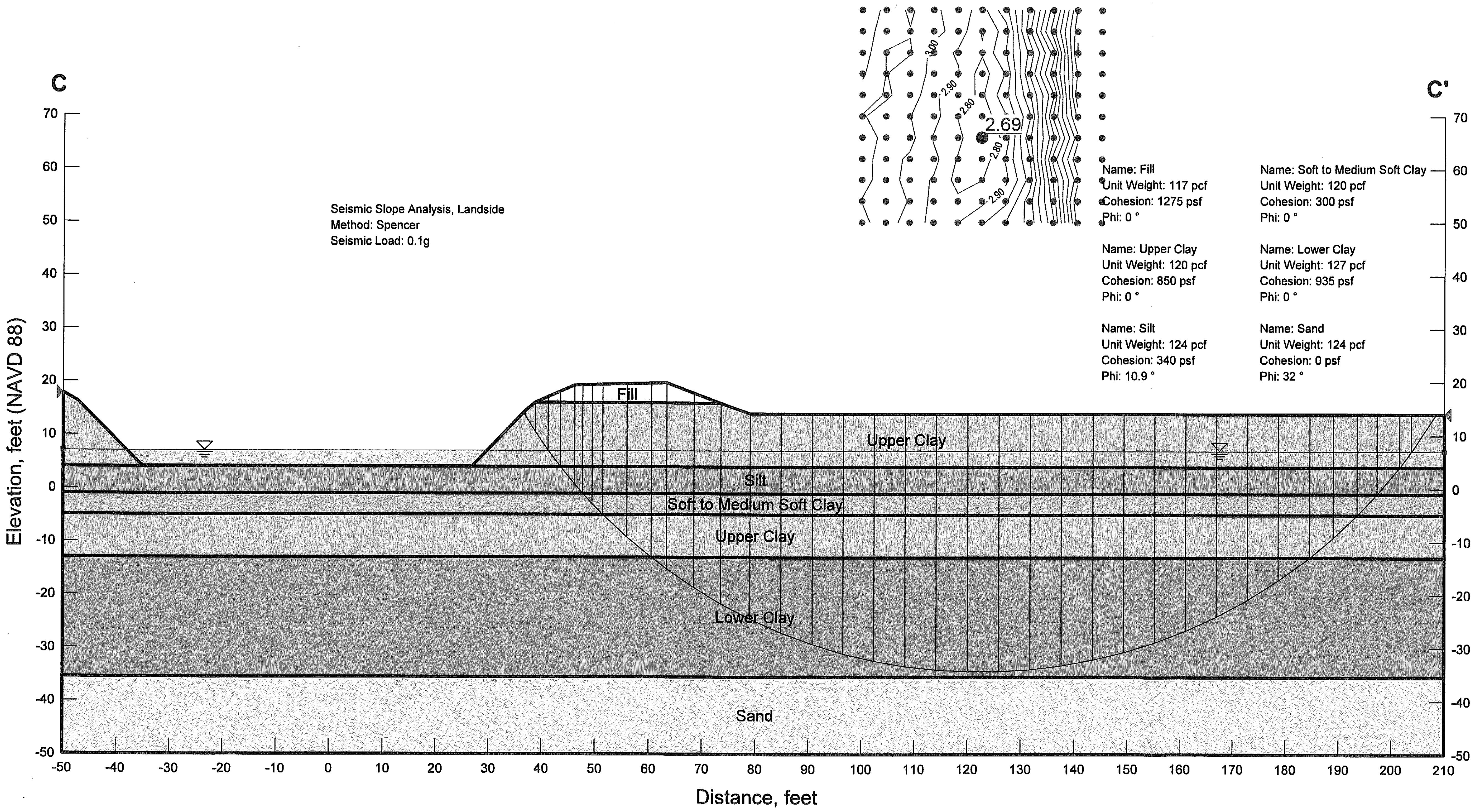
Checked

Lower Penitencia Creek
FEMA Levee Recertification
Milpitas, California

Seismic Screening Analysis
Waterside
Cross Section B
Levee Station 26+51

Figure
11-4

Directory: I:\Project\13000\13815.003\12000 CALC EVAL ANALYSES\Geo Studio\Section C\No Concrete\C_RS_4440_Seismic.gsz   Date: 7/8/2009   Time: 10:37:04 AM   Last Edited By: Levulett, Darr



Seismic Slope Analysis, Landside
Method: Spencer
Seismic Load: 0.1g

Name: Fill
Unit Weight: 117 pcf
Cohesion: 1275 psf
Phi: 0 °

Name: Soft to Medium Soft Clay
Unit Weight: 120 pcf
Cohesion: 300 psf
Phi: 0 °

Name: Upper Clay
Unit Weight: 120 pcf
Cohesion: 850 psf
Phi: 0 °

Name: Lower Clay
Unit Weight: 127 pcf
Cohesion: 935 psf
Phi: 0 °

Name: Silt
Unit Weight: 124 pcf
Cohesion: 340 psf
Phi: 10.9 °

Name: Sand
Unit Weight: 124 pcf
Cohesion: 0 psf
Phi: 32 °

amec

Scale
1 inch = 20 feet

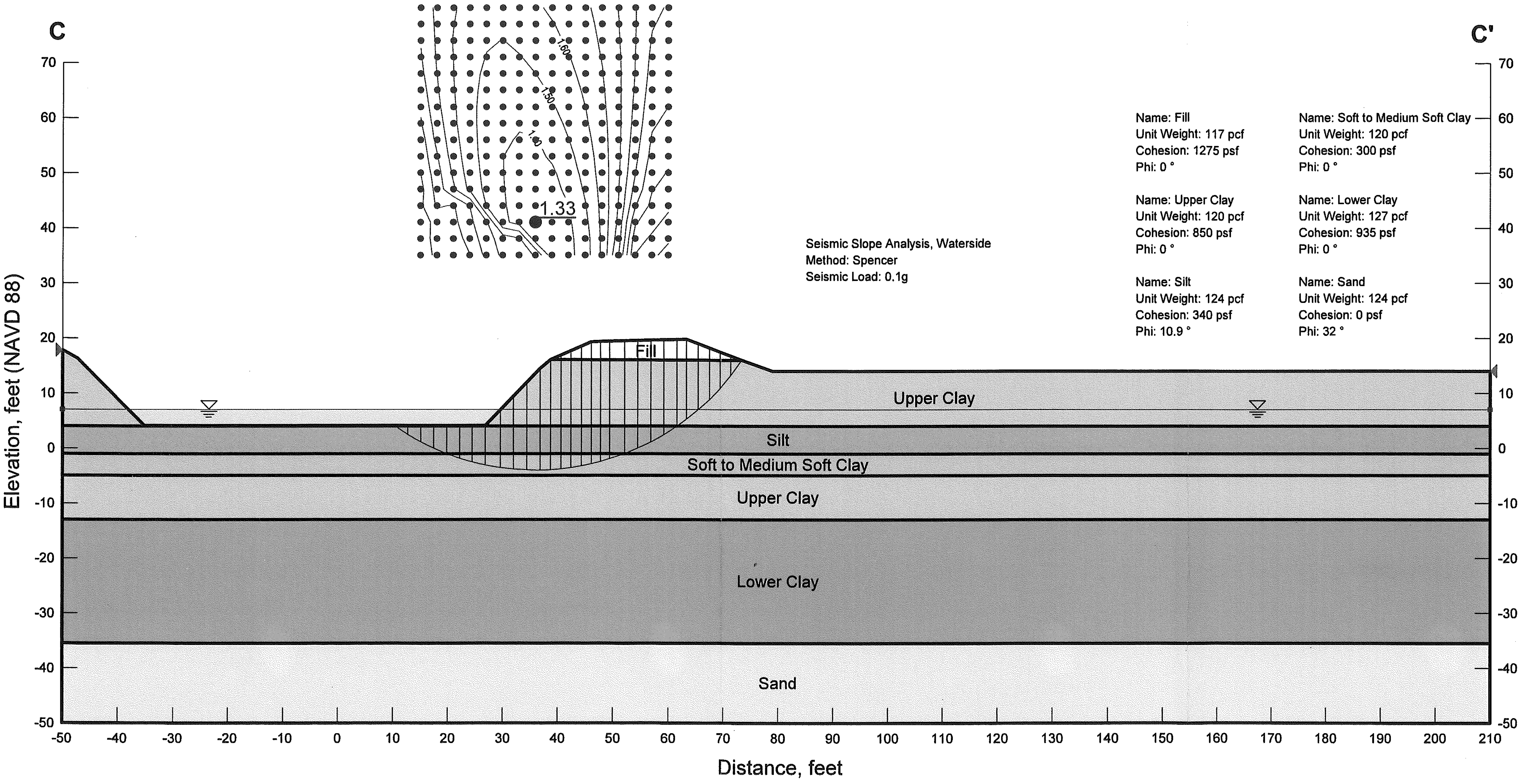
Project Number 13815.003	Drawn D. Levulett
Task 0C.03	Analyzed D. Levulett
Date 7/8/2009	Checked

Lower Penitencia Creek
FEMA Levee Recertification
Milpitas, California

Seismic Screening Analysis
Landside
Cross Section C
Levee Station 43+65

Figure
11-5

Directory: I:\Project\13000\13815.003\12000 CALC EVAL ANALYSES\Geo Studio\Section C\No Concrete\C_RS_4440_Seismic.gsz Date: 7/8/2009 Time: 10:37:04 AM Last Edited By: Levulett, Darr



amec

Scale
1 inch = 20 feet

Project Number
13815.003

Task
0C.03

Date
7/8/2009

Drawn
D. Levulett

Analyzed
D. Levulett

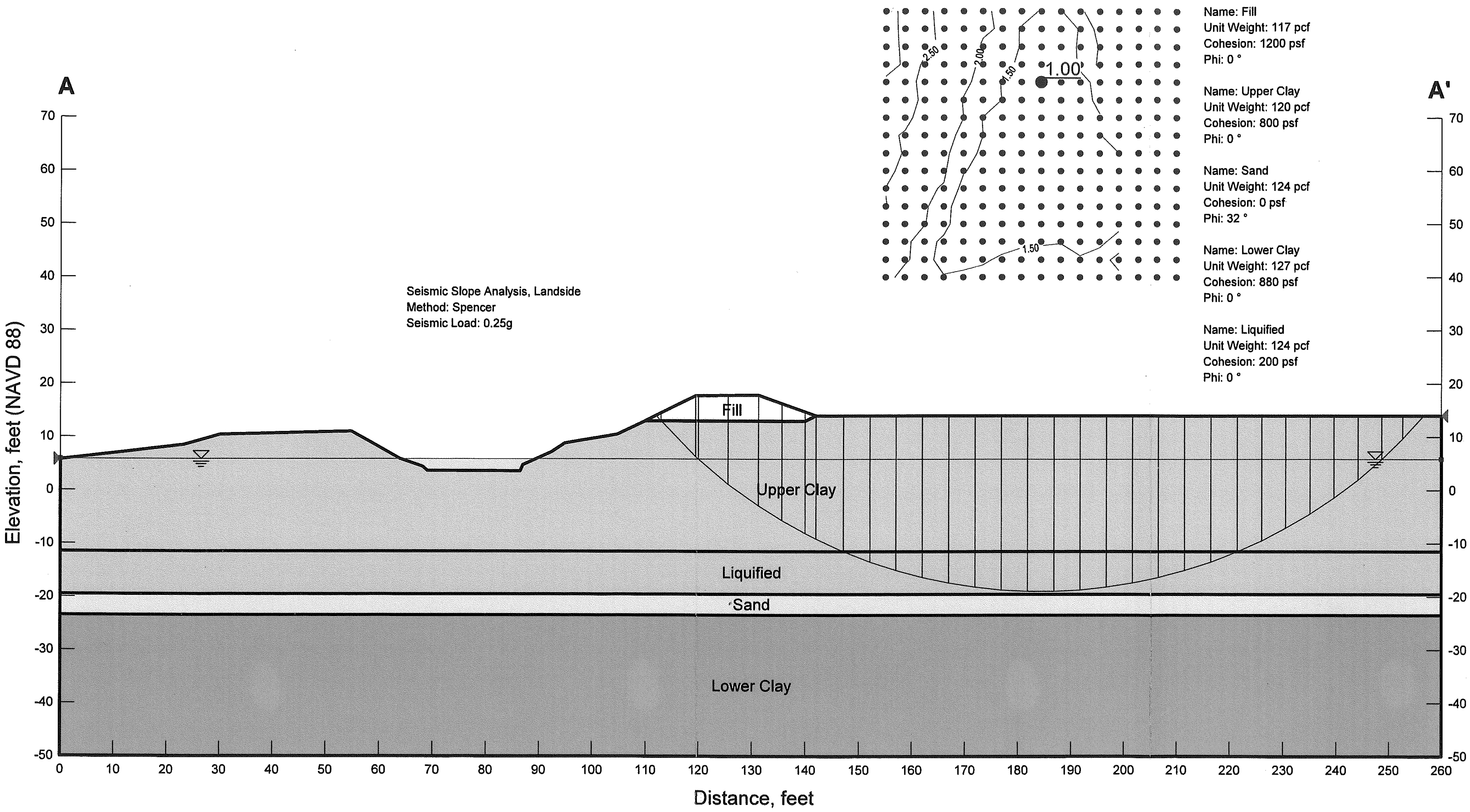
Checked JAJ

Lower Penitencia Creek
FEMA Levee Recertification
Milpitas, California

Seismic Screening Analysis
Waterside
Cross Section C
Levee Station 43+65

Figure
11-6

Directory: I:\Project\13000\13815.003\12000 CALC EVAL ANALYSES\Geo Studio\Section AA_RS_1640_SeismicKy.gsz Date: 5/11/2009 Time: 11:11:59 AM Last Edited By: Levulett, Darcie



Seismic Slope Analysis, Landside
Method: Spencer
Seismic Load: 0.25g

Name: Fill
Unit Weight: 117 pcf
Cohesion: 1200 psf
Phi: 0 °

Name: Upper Clay
Unit Weight: 120 pcf
Cohesion: 800 psf
Phi: 0 °

Name: Sand
Unit Weight: 124 pcf
Cohesion: 0 psf
Phi: 32 °

Name: Lower Clay
Unit Weight: 127 pcf
Cohesion: 880 psf
Phi: 0 °

Name: Liquified
Unit Weight: 124 pcf
Cohesion: 200 psf
Phi: 0 °

amec	**Scale** 1 inch = 20 feet	Project Number 13815.003	Drawn D. Levulett	**Lower Penitencia Creek** **FEMA Levee Recertification** **Milpitas, California**	**Seismic Yield Determination** Landside Cross Section A Levee Station 15+65	**Figure** 12-1
		Task 0C.03	Analyzed D. Levulett			
		Date 5/11/2009	Checked PSK			

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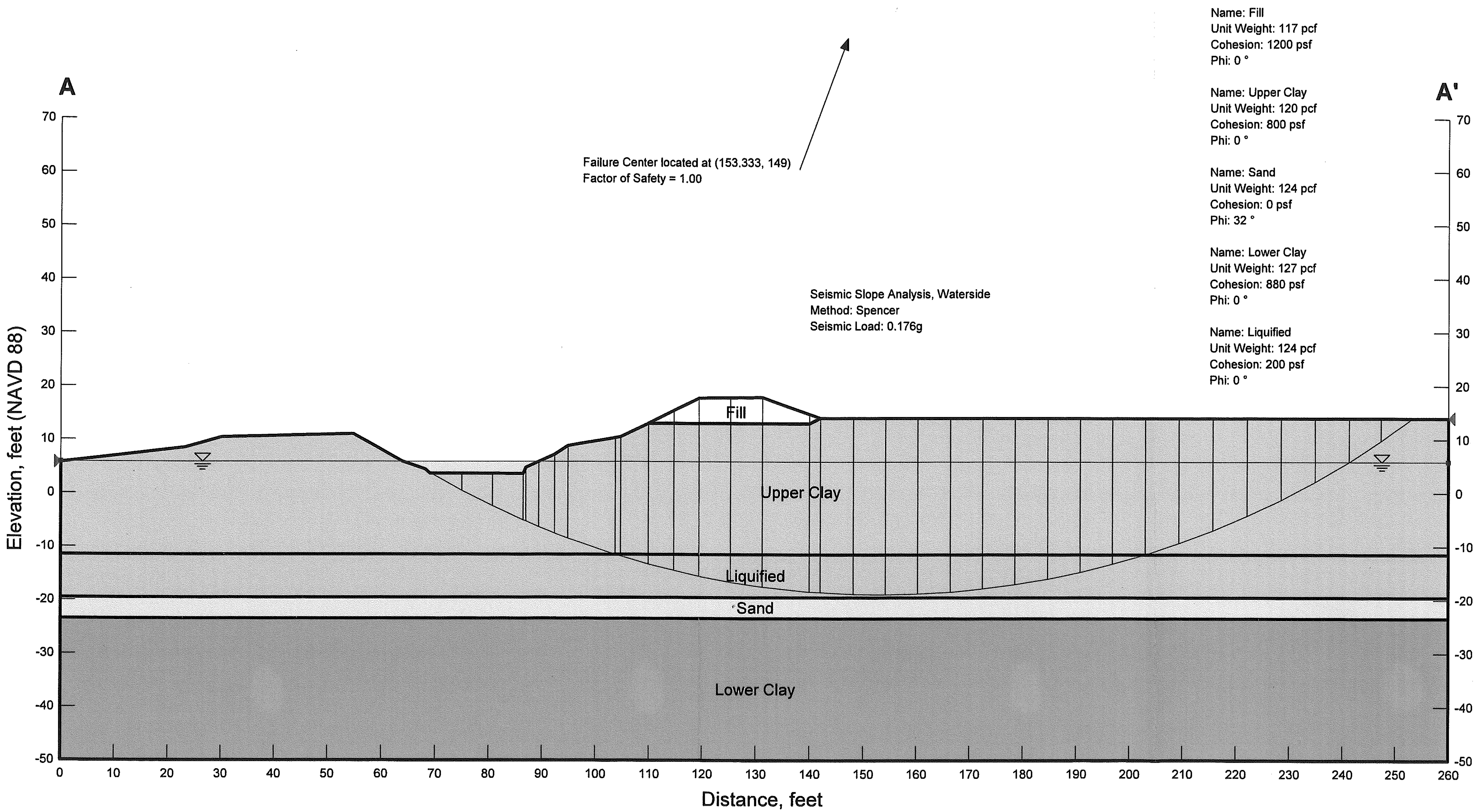


amec

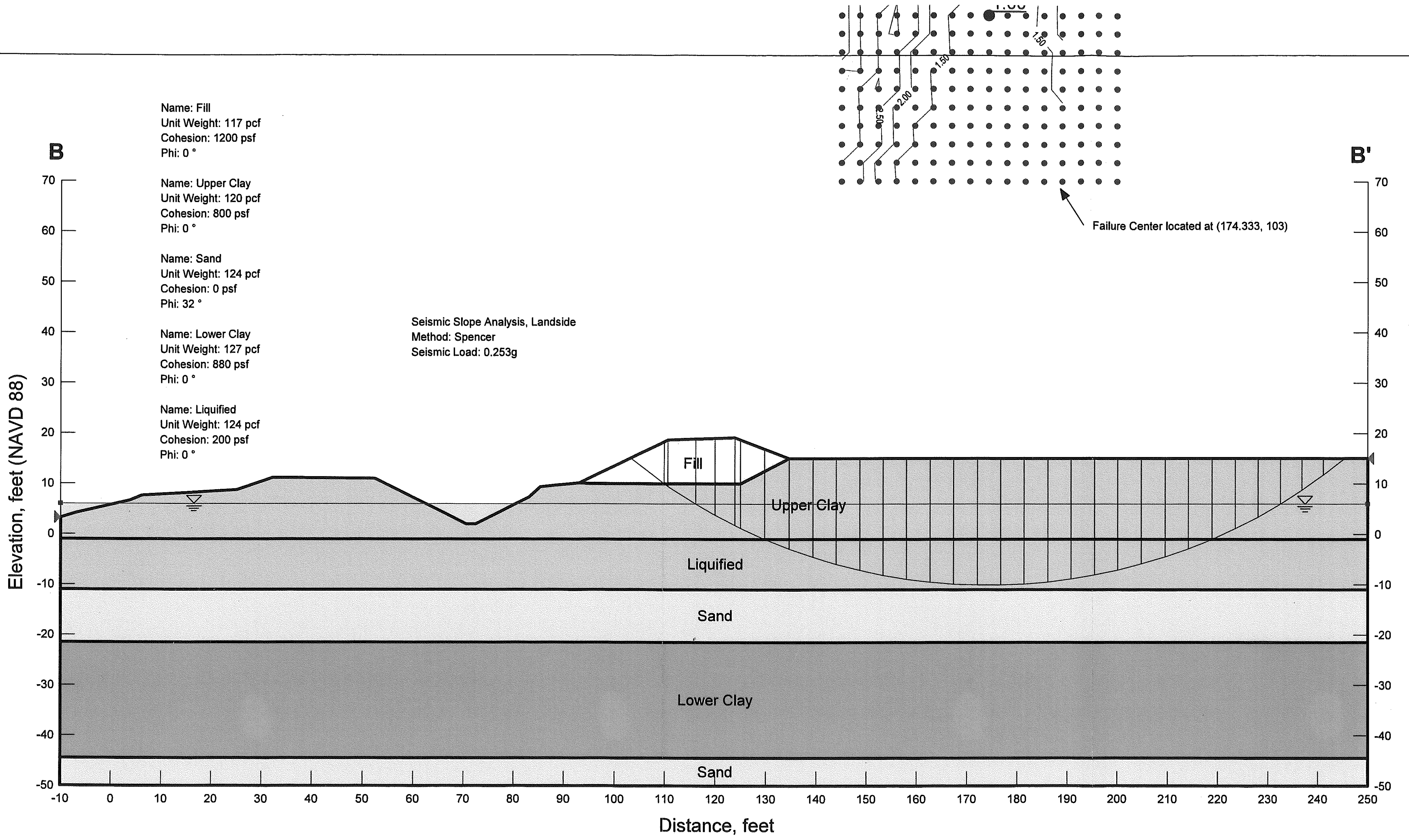
Scale 1 inch = 20 feet	Project Number 13815.003	Drawn D. Levulett	
	Task 0C.03	Analyzed D. Levulett	
	Date 5/11/2009	Checked psk	

Lower Penitencia Creek
FEMA Levee Recertification
Milpitas, California

Seismic Screening Analysis
Waterside
Cross Section A
Levee Station 15+65

Figure
12-2

Directory: I:\Project\13000\13815.003\12000 CALC EVAL ANALYSES\Geo Studio\Section B\B_RS_3026_SeismicKy.gsz Date: 5/21/2009 Time: 10:19:34 AM Last Edited By: Levulett, Darcie



amec

Scale
1 inch = 20 feet

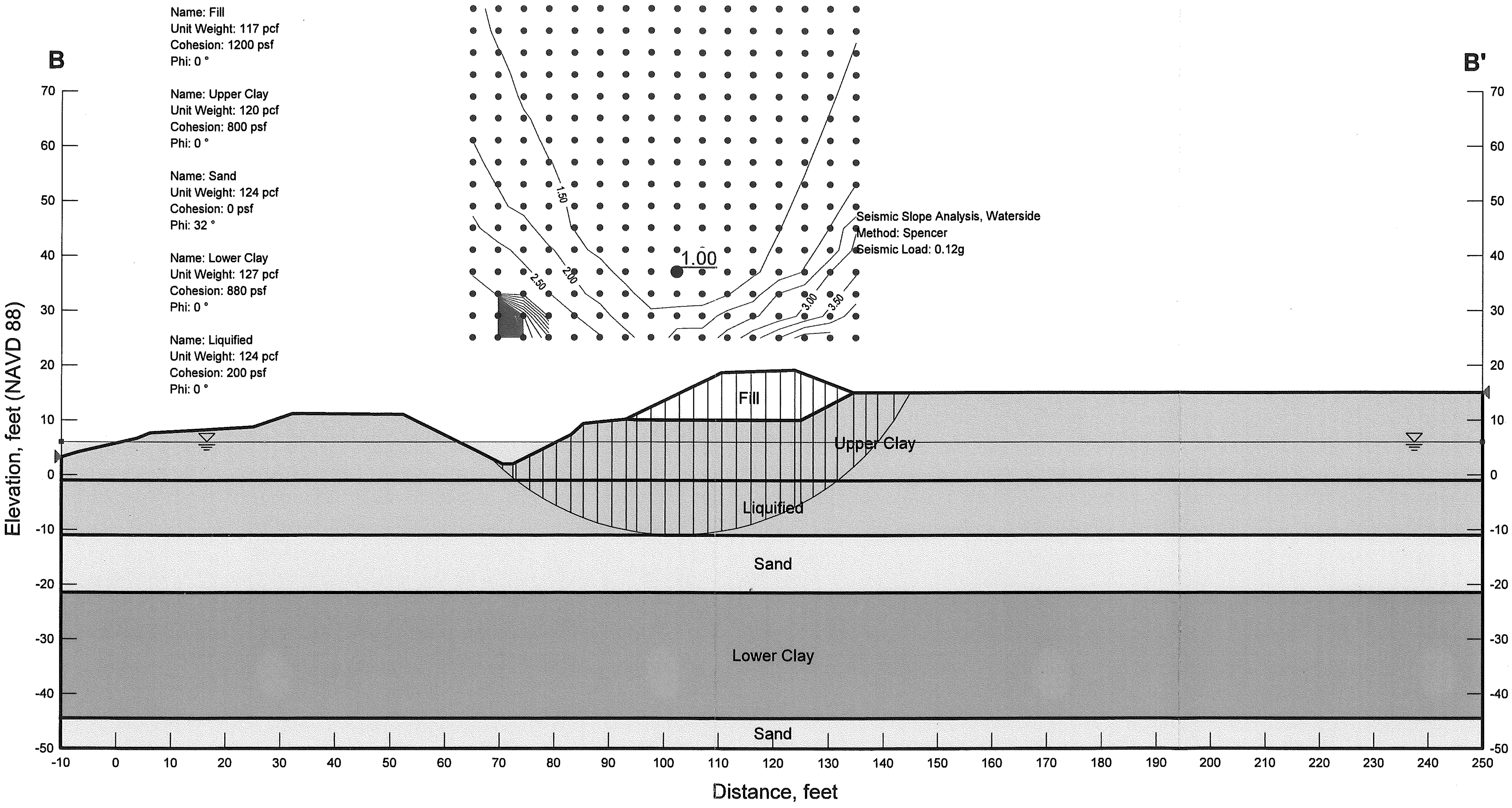
Project Number 13815.003	Drawn D. Levulett
Task 0C.03	Analyzed D. Levulett
Date 5/21/2009	Checked PSK

Lower Penitencia Creek
FEMA Levee Recertification
Milpitas, California

Seismic Yield Determination
Landside
Cross Section B
Levee Station 26+51

Figure
12-3

Directory: I:\Project\13000\13815.003\12000 CALC EVAL ANALYSES\Geo Studio\Section B\B_RS_3026_SeismicKy.gsz Date: 5/21/2009 Time: 10:24:31 AM Last Edited By: Levulett, Darcie



amec

Scale
1 inch = 20 feet

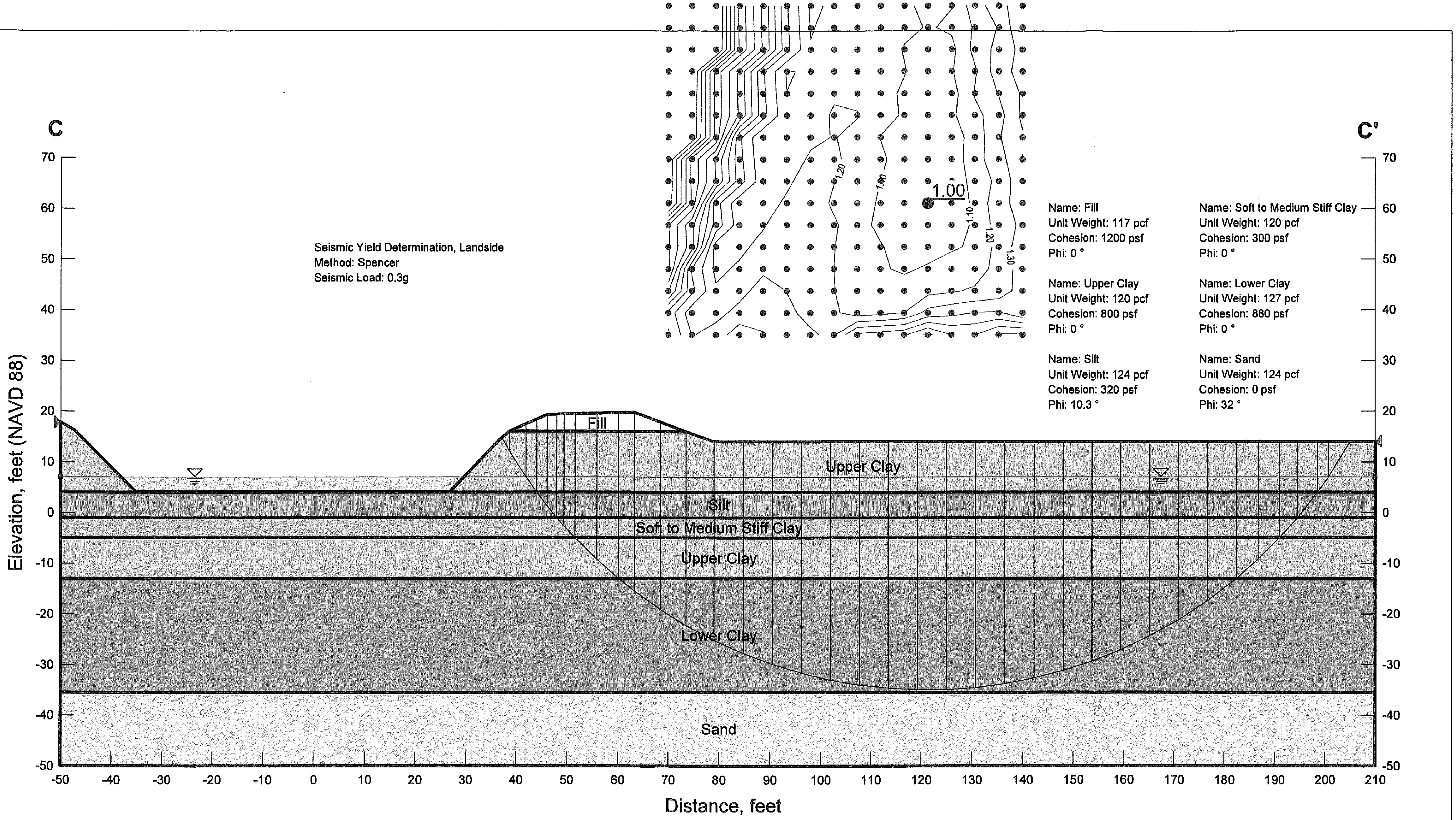
Project Number 13815.003	Drawn D. Levulett
Task 0C.03	Analyzed D. Levulett
Date 5/21/2009	Checked PSK

Lower Penitencia Creek
FEMA Levee Recertification
Milpitas, California

Seismic Yield Determination
Waterside
Cross Section B
Levee Station 26+51

Figure
12-4

Directory: I:\Project\13000\13815.003\12000 CALC EVAL ANALYSES\Geo Studio\Section C\No Concrete\C_RS_4440_SeismicKy_soft.gsz Date: 7/8/2009 Time: 10:40:41 AM Last Edited By: Levul



amec

Scale
1 inch = 20 feet

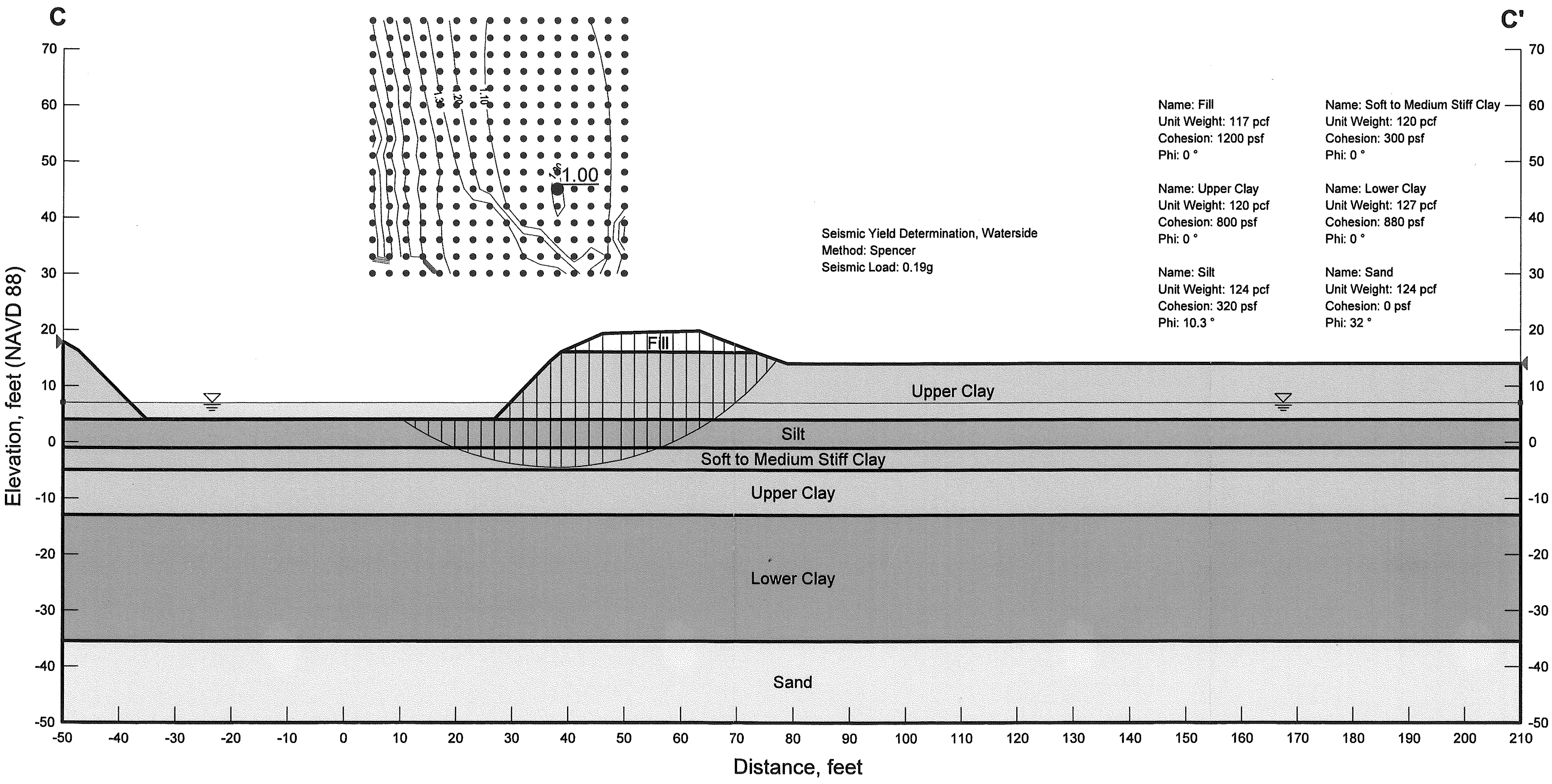
Project Number 13815.003	Drawn D. Levulett
Task 0C.03	Analyzed D. Levulett
Date 7/8/2009	Checked JAT

Lower Penitencia Creek
FEMA Levee Recertification
Milpitas, California

Seismic Yield Determination
Landside
Cross Section C
Levee Station 43+65

Figure
12-5

Directory: I:\Project\13000\13815.003\12000 CALC EVAL ANALYSES\Geo Studio\Section C\No Concrete\C_RS_4440_SeismicKy_soft.gsz Date: 7/8/2009 Time: 10:40:41 AM Last Edited By: Levult



Scale	Project Number 13815.003	Drawn D. Levulett	Lower Penitencia Creek FEMA Levee Recertification Milpitas, California	Seismic Yield Determination Waterside Cross Section C Levee Station 43+65	Figure 12-6
1 inch = 20 feet	Task 0C.03	Analyzed D. Levulett			
	Date 7/8/2009	Checked			

APPENDIX A

APPENDIX A

Previous Data

ATTACHMENT A.1 Logs of Borings by Reimer Associates (1991)

APPENDIX A

DATA REVIEW

Lower Penitencia Creek Levee Recertification
Milpitas, California

As part of our evaluation of subsurface conditions for the subject levees, AMEC compiled and reviewed geologic and geotechnical information in the vicinity of the subject levees contained in our files and the archives of the Santa Clara Valley Water District. The site map and boring logs from relevant previous subsurface investigations are provided for reference and included in this appendix. The locations or previous explorations are also indicated on Figures 2 and 3 of this report.

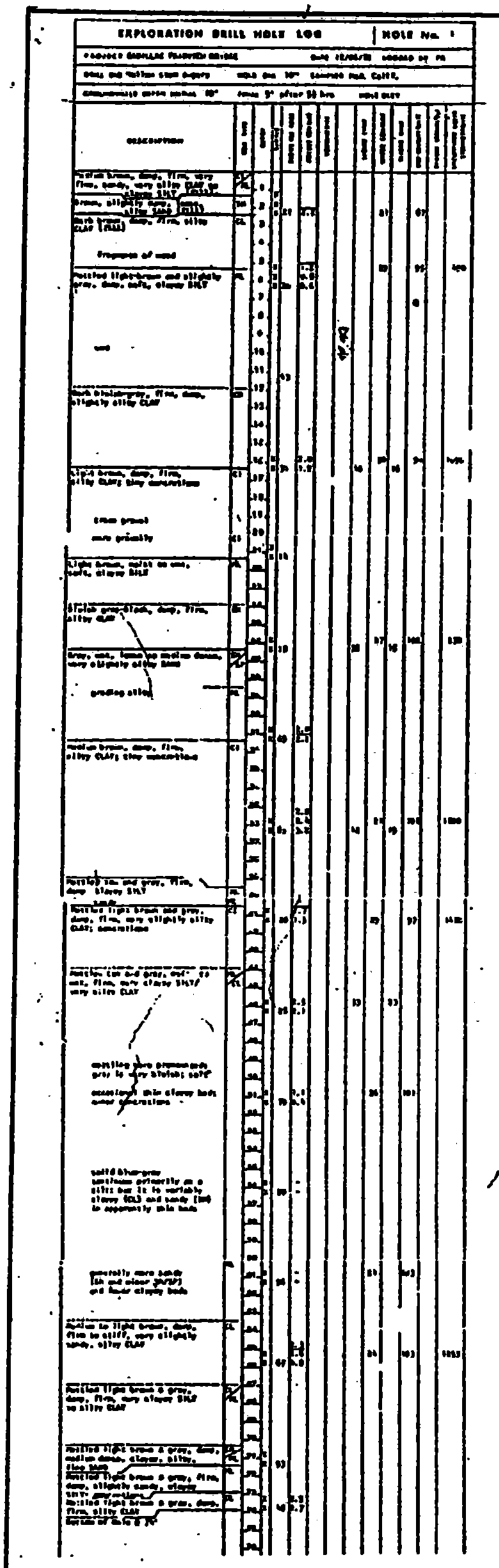
It should be noted that the files we reviewed may not include all the geologic/geotechnical investigation reports prepared for projects in the vicinity of the subject levees. In addition, this report does not include all the information for the study area that is available in the archives reviewed by AMEC. Only the information we judged to be most pertinent to the present study is included in this appendix.

Because the information presented in this appendix does not completely describe the evaluation and exploration techniques used or the subsurface and groundwater conditions encountered, the reader may want to review the original reports from which the information was excerpted. The logs included in this appendix should be considered to depict subsurface conditions only at the specific locations and at particular times the exploration work was performed and/or water levels were measured. Soil and groundwater conditions at other locations may differ from the conditions occurring at these locations. Also, the passage of time may result in changes in the soil and groundwater conditions at these locations.

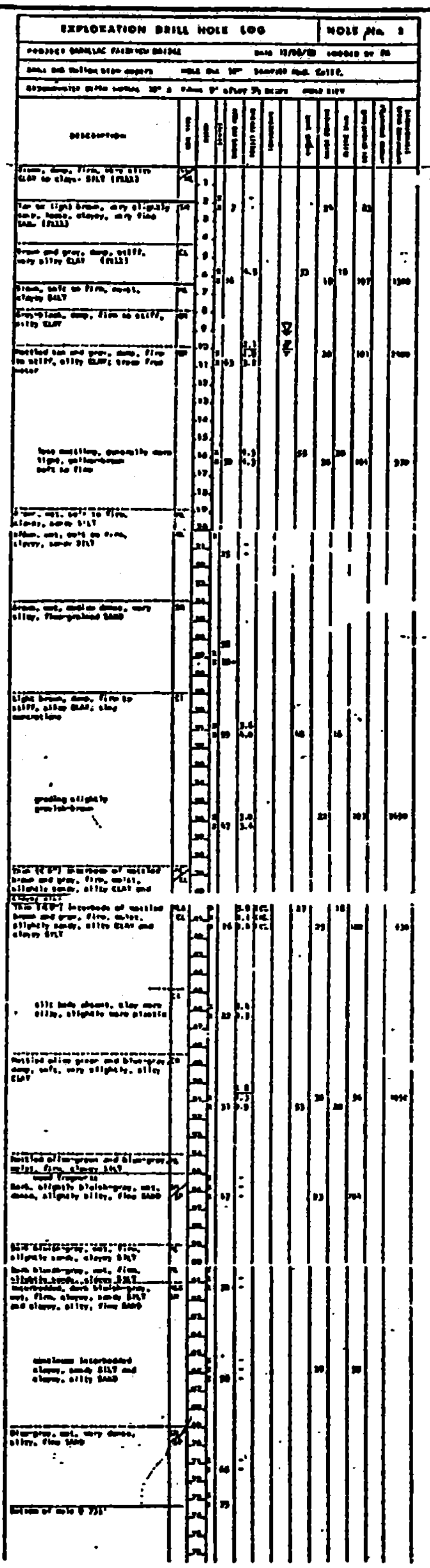
ATTACHMENT A.1

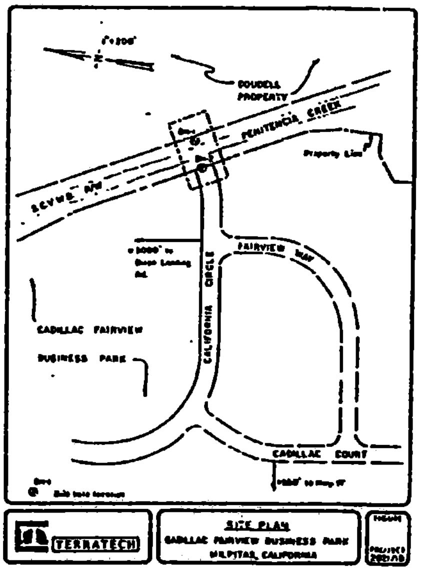
Logs of Borings by Reimer Associates (1991)

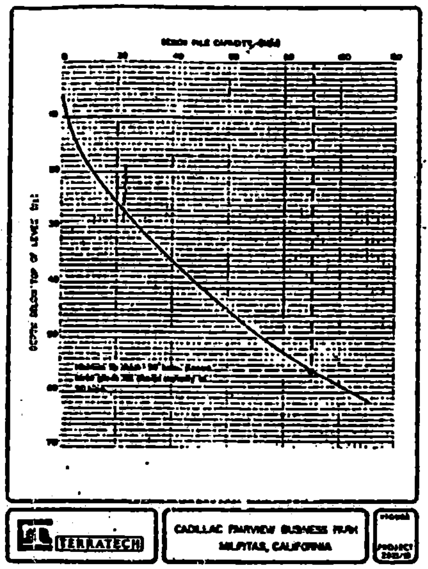
EXPLORATION DRILL HOLE LOG | **HOLE No. 1**

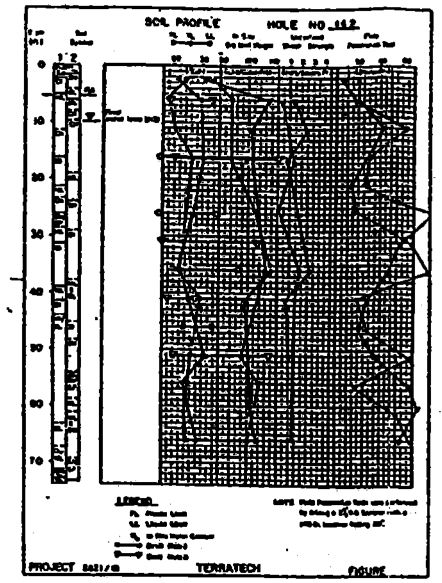


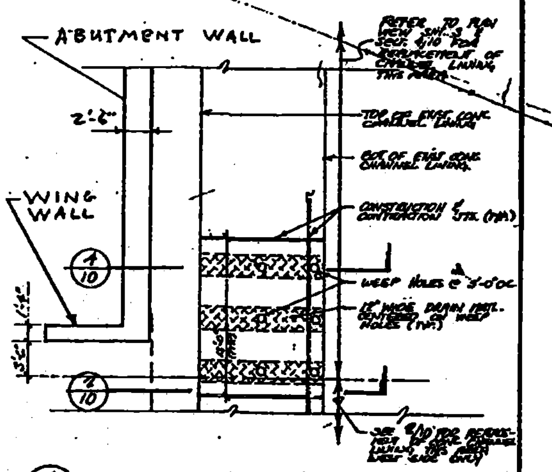
EXPLORATION DRILL HOLE LOG | **HOLE No. 2**





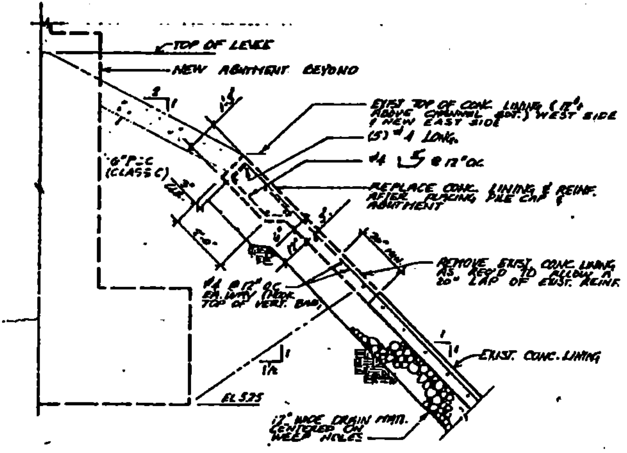





ABUTMENT WALL
WING WALL
2'-6"
REFER TO PLAN VIEW SHT. 3 & SECT. 4/10 FOR REPLACEMENT OF CHANNEL LINING THIS AREA
TOP OF EXIST. CONC. CHANNEL LINING
BOT. OF EXIST. CONC. CHANNEL LINING
CONSTRUCTION & CONTRACTION JTS. (TYP.)
WEEP HOLES @ 5'-0" O.C.
12" WIDE DRAIN MATL. CENTERED ON WEEP HOLES (TYP.)
SEE 2/10 FOR REPLACEMENT OF CONC. CHANNEL LINING THIS AREA EAST SIDE ONLY
4/10
2/10
1'-4"
3'-6"

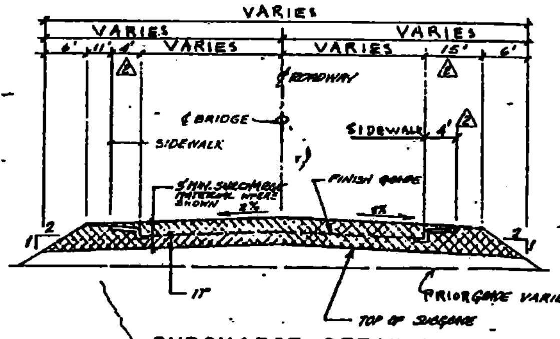
1/10 PLAN VIEW (Partial)-CHANNEL LINER

SCALE: 1/8"=1'-0"


TOP OF LEVEE
NEW ABUTMENT BEYOND
EXIST TOP OF CONC. LINING (12'± ABOVE CHANNEL BOT.) WEST SIDE & NEW EAST SIDE
(5) #4 LONG.
#4 @ 12" OC
REPLACE CONC. LINING & REINF. AFTER PLACING PILE CAP & ABUTMENT
6" P.C.C. (CLASS C)
REMOVE EXIST. CONC. LINING AS REQ'D TO ALLOW A 20" LAP OF EXIST. REINF.
#4 @ 12" OC EA. WAY (HOOK TOP OF VERT. BAR)
EXIST. CONC. LINING
EL 52.5
12" WIDE DRAIN MATL. CENTERED ON WEEP HOLES
2
1
1½
20" MIN.
5'-0"

2/10 SECTION

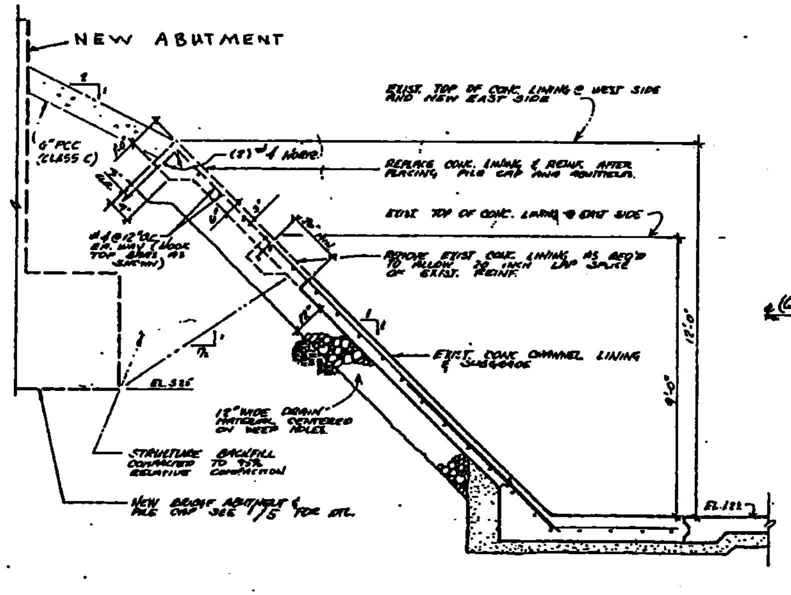
SCALE: 3/8"=1'-0"



SURCHARGE DETAIL

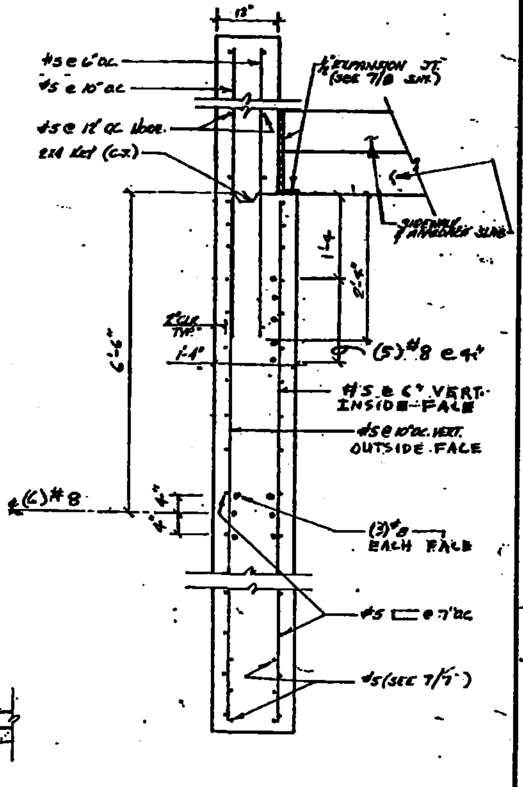
3/10 TYPICAL SECTION - STA. 45+50 to 47+63.9 & STA. 48+90.7 to 50+00

N.T.S.


NEW ABUTMENT
EXIST. TOP OF CONC. LINING @ WEST SIDE AND NEW EAST SIDE
6" PCC (CLASS C)
(7) #4 HORIZ.
REPLACE CONC. LINING & REINF. AFTER PLACING PILE CAP AND ABUTMENT.
EXIST. TOP OF CONC. LINING @ EAST SIDE
#4 @ 12" OC EA. WAY (HOOK TOP BARS AS SHOWN)
REMOVE EXIST. CONC. LINING AS REQ'D TO ALLOW 20 INCH LAP SPLICE OF EXIST. REINF.
EXIST. CONC. CHANNEL LINING & SUBGRADE
EL 52.5
12" WIDE DRAIN MATERIAL CENTERED ON WEEP HOLES
STRUCTURE BACKFILL COMPACTED TO 95% RELATIVE COMPACTION
NEW BRIDGE ABUTMENT & PILE CAP SEE 1/5 FOR DTL.
EL 12.2
12'-0"
9'-0"

4/10 SECTION

SCALE: 3/8"=1'-0"



5/10 SECTION THRU WINGWALL

SCALE: 3/8"=1'-0"

No. 784
JOHN WEBSTER BROWN

JOHN WEBSTER BROWN / CIVIL & STRUCTURAL ENGINEERS
1135 Terminal Way — Suite 109 / Reno, Nevada 89502 / (702) 322-3878

DATE: 8/13/86					
SCALE: AS SHOWN					
DESIGNED: JWB, CDG, RPF					
DRAWN: RPF					
CHECKED: JWB	2		2/13/87	DIMENSIONS - SECTION 3	
PROJ. ENGR. JWB	1	JWB	1/15/87	68' WIDTH	
		BY	DATE	REVISIONS	

APPROVED
BY: __________
DATE: ________ R.C.E. ________

APPROVED
BY: __________
DATE: ________ R.C.E. ________

Reimer Associates
URBAN / ENVIRONMENTAL SYSTEMS ENGINEERS
120 Burlingame Office Center • 1633 Old Bayshore Highway
Burlingame, California • 94010 • (415) 692-1830

BORING LOG & MISC. DETAILS
IMPROVEMENT PLANS
MILPITAS CALIFORNIA

SANTA CLARA_CA Work Maps
SANTA CLARA_CA0861

SHEET 10 OF 18 SHEETS
JOB NO. R-958

AS-BUILT

AS-BUILT 2-755

JWB 1986 R98

1-2

APPENDIX B

APPENDIX B

Field Reconnaissance Memorandum

amec

Memo

TO:	Chuck Anderson/S&W	**DATE:**	June 12, 2009
FROM:	Darcie Levulett Jim French	**PROJ. NO.:**	13815.003.0
CC:	Liza McNulty/S&W Beeson Liang/PGE	**PROJ. NAME:**	Lower Penitencia Creek Levee Recertification

SUBJECT: **Summary of Geotechnical Field Reconnaissance – Lower Penitencia Creek East Bank Levee, Milpitas, California**

INTRODUCTION

The Santa Clara Valley Water District (District) has initiated a project to evaluate and recertify District-owned levees in accordance with Federal Emergency Management Agency (FEMA) flood mapping standards. To qualify for recertification, the levees must be shown to conform to the requirements of Title 44 of the Code of Federal Regulations (44 CFR), Section 65.10, which includes minimum certification standards for levee design, operations, and maintenance.

The District has retained Schaaf & Wheeler (S&W), with AMEC Geomatrix, Inc. (AMEC; formerly known as Geomatrix Consultants, Inc.) and Pacific Geotechnical Engineering (PGE) as geotechnical subconsultants, to perform topographic surveying, geotechnical exploration, laboratory testing, and hydraulic and engineering analyses, calculations, and evaluations necessary to recertify the existing Lower Penitencia Creek east bank levee in Milpitas, California. Upon completion of these efforts, certification documents will be prepared by S&W and submitted to FEMA by the District for accreditation and incorporation of the levees into appropriate Flood Insurance Rate Maps (FIRMs).

As an initial step in the recertification process, AMEC reviewed available information and performed geotechnical field reconnaissance of a section of the Lower Penitencia Creek east bank levee that extends from Berryessa Creek to the California Circle Bridge, a distance of approximately 4,000 feet. The purpose of the reconnaissance was to observe levee conditions and identify areas for further investigation and study. Reconnaissance efforts were primarily focused on identifying geotechnical issues of concern, including potential settlement, slope instability, and seepage problems. However, other related considerations also were observed and documented as part of the reconnaissance, including the presence and condition of closures and penetrations; erosion; encroachments by vegetation, utilities, and structures; and maintenance considerations.

This technical memorandum presents the findings of AMEC's field reconnaissance and discusses key observations, geotechnical issues of concern, and other issues of potential concern related to the proposed recertification of the Lower Penitencia Creek east bank levee between Berryessa Creek and the California Circle Bridge. Also, this memorandum identifies areas and conditions that require further investigation and/or study and suggests possible maintenance solutions that may be implemented to mitigate potential problems encountered during the reconnaissance.

RECONNAISSANCE ACTIVITIES

Field reconnaissance of the levee along the east bank of Lower Penitencia Creek was performed by Mr. David Ritzman (California GE #2710), a Senior Engineer with AMEC, on August 12 and 13, 2008. Reconnaissance activities included the following:

- visually observing the levee, creek channel, and adjacent areas for evidence of settlement, erosion, ground deformation/cracking, and potential stability problems
- approximately measuring levee dimensions and slope inclinations
- identifying the presence and condition of erosion protection measures, outfalls, closures, floodwalls, and other structures and improvements along the levee alignment
- identifying trees, utilities, and other features that encroach on levee slopes
- observing water levels and flow conditions in the creek channel
- photographing the levee, adjacent areas, and other features of interest

Conditions observed during the field reconnaissance were documented in levee field reconnaissance logs, copies of which are included as Attachment 1 to this memorandum. The field logs also include references to approximate project stationing, as well as brief descriptions of photographs collected during the reconnaissance effort. Photographs, along with accompanying descriptions and location references, are presented in Attachment 2.

Locations of observations, photographs, and other features described in the field reconnaissance logs and in this memorandum are referenced to the District's levee stationing system. Most of the study area is referenced to the Penitencia Creek control line, which begins approximately 1,100 feet west of the study area, at the confluence of Penitencia and Coyote Creeks, and proceeds along the creek channel in the upstream direction. An approximately 200-foot-long section of the levee at the southern end of the study area is referenced to the Berryessa Creek control line. Aerial photographs of the levee, along with the District's stationing lines, are shown on Figures 1 and 2. In the field, positions relative to the stationing lines were estimated based on the locations of features visible on the aerial photographs. Distances were measured using a measuring wheel. Because of the inexact nature of these methods and measurements, references to the District's stationing lines presented in the field reconnaissance logs and this memorandum should be considered approximate.

LEVEE DESCRIPTION

As shown on Figures 1 and 2, the Lower Penitencia Creek east bank levee study area begins immediately upstream of the California Circle Bridge, at approximately Station 11+40. From the bridge, the levee extends approximately 3,200 feet (0.6 miles) to the confluence of Lower Penitencia and Berryessa Creeks at about Station 43+75. From Station 43+75, the levee extends an additional 200 feet along the Berryessa Creek control line to the southern end of the study area. The levee was constructed by a private developer (Reimer Associates) in 1991.

For convenience in documenting observations during our field reconnaissance, the Lower Penitencia Creek east bank levee was divided into three reaches, each with relatively uniform characteristics in regards to geometry and/or configuration. The reaches range from approximately 740 to approximately 1,880 feet in length. Table 1 summarizes the location and general characteristics of each reach. In Attachment 1, more detailed reach descriptions are provided in the field reconnaissance logs.

KEY OBSERVATIONS

During our field reconnaissance, each levee reach was visually screened for the following conditions:

- evidence of settlement (i.e., surface cracks, sinkholes, and hummocky/undulating ground)
- erosion (i.e., rills, gullies, scour around structures, and undercut or oversteepened levee and stream bank slopes)
- evidence of potential slope stability problems (i.e., steep and/or undercut slopes, surface cracks, and caving, bulging, slumping, or irregular ground surfaces)
- other ground deformation and/or cracking not obviously related to settlement, erosion or slope instability
- encroachments by vegetation or other improvements
- inadequate maintenance (i.e., mowing/vegetation control, debris/blockages in the channel, inoperable control structures, etc.)
- evidence of potential problems with walls, abutments, weirs, aprons, and other structures within the levee and channel area
- evidence of seepage (e.g., wetland-type vegetation, moist or wet ground, springs and/or boils on the outboard side of the levee)
- damaged, distressed, and/or blocked closures or interior drainage structures, including gates, flaps, and other devices for backflow control
- adverse channel conditions (i.e., blockages, excessive debris, sand/gravel bars, etc.)
- animal burrows

Where the above conditions were observed, the type, location, and severity of the condition were noted in the field reconnaissance logs. Observed conditions were subjectively assigned one of five severity levels: (1) slight, (2) slight to moderate, (3) moderate, (4) moderate to severe, and (5) severe. Each reach was screened for all of the above conditions; if a particular condition is not noted in the reach's reconnaissance log, it was not observed in the field.

Table 2 summarizes key observations made along each levee reach during the site reconnaissance. As shown in Table 2, key observations apply to one or more of the conditions described above. The severity of each condition as it relates to the observation is also presented in Table 2.

GEOTECHNICAL ISSUES OF POTENTIAL CONCERN

Geotechnical issues pertaining to levee certification include embankment and foundation stability, seepage, and settlement. For certification under FEMA standards, the following conditions must be demonstrated:

- Levee embankment and foundation stability will not be jeopardized under anticipated seepage and loading conditions associated with the base (i.e., the one-percent-annual-chance, or "100-year") flood.
- Embankment and foundation stability will not be jeopardized under anticipated seismic loading conditions.
- Potential settlement of the levee embankment will not cause the amount of freeboard to fall below required minimum values.

For existing levees, recertification efforts could be jeopardized by conditions that exhibit evidence of or indicate the potential for embankment and foundation instability, adverse seepage conditions, and/or excessive settlement.

In our opinion, the levee and channel conditions observed during field reconnaissance of the Lower Penitencia Creek east bank levee do not generally indicate serious geotechnical problems that could jeopardize potential levee recertification. However, a number of minor geotechnical issues of potential concern were noted during the reconnaissance. This section discusses conditions observed along the Lower Penitencia Creek east bank levee that directly and indirectly relate to stability, seepage, and settlement. Recommended approaches for additional investigation, study, and maintenance in these areas are also described, as appropriate.

Steep Levee and Stream Bank Slopes

Relatively steep levee and stream bank slopes present potential concerns for levee embankment and foundation stability. Although stability is a function of many factors, including slope height and properties of the underlying soil, steep slopes typically are more susceptible to instability than shallow slopes. If underlain by expansive soil, relatively steep slopes may also be subject to slope creep, a gradual downslope movement of near-surface soil in response to seasonal shrink-swell cycles.

During our field reconnaissance, levee and stream bank slopes steeper than approximately 2:1 (about 27 degrees) were identified for special consideration as to potential instability. The locations of these relatively steep slopes are summarized in Table 2. In general, steeper slope conditions were assigned a higher degree of severity, particularly where accompanied by other evidence of potential instability, such as severe erosion, ground deformation, and surface cracking.

A geotechnical investigation consisting of subsurface exploration and laboratory testing should be performed to identify and characterize soil and groundwater conditions in the vicinity of relatively steep levee and stream bank slopes. Information collected during the investigation should be used to develop appropriate soil parameters for evaluating the stability of existing levee and stream bank slopes under FEMA-designated loading conditions.

Concrete Lined Slopes

Moderately steep to very steep concrete-surfaced inboard and outboard levee slopes were observed during our field reconnaissance at electrical tower locations near Stations 21+30, 29+10, and 37+00. At these locations, a pedestrian path runs between the electrical towers and the toe of the outboard levee slope. To allow space for the path, outboard levee slopes become steeper approximately 100 feet upstream and downstream of each electrical tower. In areas around the electrical towers, the concrete-surfaced outboard levee slopes are approximately 7 to 9 feet high and range from about 2:1 (27 degrees) to 1:1 (45 degrees). Approximately 50 feet upstream and downstream of each electrical tower, retaining walls with heights ranging from approximately 0.5 to 4.5 feet are located along the toe of the outboard slope. Approximately 1:1 (45 degree) slopes above the retaining walls are lined with concrete.

At tower locations, the crest of the levee shifts approximately 7 feet toward the creek channel. To accommodate this change in geometry, the inboard levee slope becomes steeper approximately 100 feet upstream and downstream of the tower location. Inboard slopes in these steepened areas are approximately 10 feet high, with slopes that range from approximately 2:1 (27 degrees) to 1.5:1 (33 degrees), and are concrete-surfaced from the crest of the levee to the edge of the lower maintenance road.

The channel and lower portion of the inboard slope is lined with concrete between Stations 37+60 (Penitencia control line) to 2+00 (Berryessa control line). The channel slopes are approximately 1:1 (about 45 degrees). The lined slope prevents erosion of the levee slope, but sediment has built up in this section and appears to have clogged drain holes near the base of the concrete channel wall.

Steep, concrete-lined slopes have stability issues similar to those described above for soil slopes. In addition, the concrete-surfaced inboard and outboard slopes near the electrical towers, with the exception of outboard retaining wall locations, do not include drains or weep holes to relieve excess pore pressure that may develop behind the concrete. The lined concrete channel between Stations 37+60 and 2+00 has weep holes to dissipate excess pore pressure, but many of the holes appear to be blocked by sediment. Excess pore pressure behind the concrete slopes may cause instability of the soil slope and lead to failure of the channel structure.

Ground Deformation and Cracking

During our field reconnaissance, cracks in the ground and pavement were observed along the crest of the levee between Stations 11+40 to 16+50, and between Stations 42+20 and 0+90. The lower portion of the levee bank slope and lower maintenance road is rutted in some areas from Station 18+80 to 37+60 and appears to be from vehicle traffic. Specific locations and descriptions of ground and pavement cracks observed during our field reconnaissance are summarized in Table 2.

A ground crack was observed on the lower inboard levee slope near Station 30+50. The crack was observed to be approximately 5 feet long and filled with soil. There appeared to be as much as approximately 3 inches of vertical offset along the crack. The lower portion of the inboard levee slope in this area has a hummocky appearance, which may indicate potential issues related to slope instability or the presence of expansive foundation materials.

Ground deformation and cracking may be the result of many factors, including expansive soil, slope creep, slope instability, and settlement. Pavement may also crack as a result of excessive loading, thermal expansion, variation in subgrade materials and properties, and natural wear and aging. Because ground and pavement cracks have a variety of potential causes, they are not necessarily indicative of potential embankment and/or foundation instability. However, in identifying areas of potential concern for slope instability during the site reconnaissance, higher levels of severity were assigned to relatively wide, deep, continuous, and/or densely spaced cracks; cracks showing evidence of vertical offset; arcuate crack patterns; ground cracks; and cracks located adjacent to steep slopes. Ground deformation and cracking patterns consistent with significant settlement of levee embankments were not observed during our field reconnaissance.

Areas of significant ground deformation and cracking identified during our field reconnaissance should be further studied to examine potential causes and evaluate whether the cracks are due to marginal stability, incipient instability, and/or slope creep. Additional study may include geotechnical investigation to characterize subsurface conditions and identify strength and shrink-swell properties of embankment and underlying foundation materials. If the observed cracks are potentially related to slope instability, stability analyses based on subsurface information collected during the geotechnical investigation may be necessary.

As part of regular maintenance activities, levee and stream bank slopes should be observed for ground deformation and cracking that may indicate potential settlement or instability problems. If the observed deformation and cracking are the result of settlement or instability, appropriate measures should be implemented to mitigate the effects of ground movement and stabilize levee slopes. Open cracks should be filled or sealed to limit infiltration of surface water and reduce the potential for continued deformation and crack growth.

Erosion and Scour

Erosion and scour within the stream channel have the potential to undermine levee and stream bank slopes and affect the stability of levee embankments and/or foundations. Areas of significant erosion and scour observed during our field reconnaissance are summarized and described in Table 2. In general, the observed erosion and scour are relatively minor to moderate. One area where the erosion that may affect the potential stability of nearby slopes is near station 28+00 where a small erosion gully and slide feature are located on the inboard levee slope.

Further study of potential hydraulic conditions within the stream channel should be performed to identify areas where erosion and scour may cause oversteepening and/or undermining of stream bank or levee slopes. Where such conditions currently exist or may occur as a result of anticipated stream flows, the need for erosion control measures to protect levee and stream bank slopes should be evaluated. As described above, geotechnical investigation may be necessary in areas where erosion has led to the development of steep levee or stream bank slopes that require analyses to evaluate their stability under FEMA-designated loading conditions.

Existing erosion control measures within the stream channel, including concrete lining, should be regularly inspected and maintained to limit the potential for instability of levee and stream bank slopes. Healthy and continuous vegetative cover should be maintained on all exposed levee and stream bank slopes. Where erosion- and scour-related damage occurs, affected slopes and structures should be repaired and appropriate erosion control measures installed to protect against further damage.

Animal Burrows

Animal burrows may weaken levee and stream bank slopes and provide potential conduits for seepage and/or piping beneath and through levees. Burrow holes and associated piles of loose soil material also disturb the ground surface and may contribute to localized erosion of levee and stream bank slopes. Ground squirrel burrows can grow to 30 feet or more in length and extend 2 to 4 feet below the ground surface. Gopher burrows are typically shallower, but can extend for several hundred feet.

During our field reconnaissance, burrow holes less than about 2 to 3 inches in diameter were attributed to gophers, and burrow holes greater than about 2 to 3 inches in diameter were attributed to ground squirrels, although in most cases the actual type of burrowing animal that created the hole is uncertain. Because of their larger diameter and presumed depth, ground squirrel burrows observed in levee and stream bank slopes were typically assigned higher levels of relative severity than gopher burrows.

Areas of significant burrowing activity that were observed during our field reconnaissance are summarized and described in Table 2. In general, the severity of burrowing observed on the levee is slight.

During our field reconnaissance, heavy vegetation may have obscured evidence of burrowing activity on some levee embankment and stream bank slopes. A more thorough burrow survey, conducted at a time when the vegetation is lower, may be considered as part of further studies. Burrow surveys should be performed regularly as part of ongoing levee maintenance programs. Where significant burrowing activity is observed, appropriate measures to control gopher and/or ground squirrel populations, such as trapping and baiting, should be implemented. Levee embankments and foundations affected by burrowing activity should be repaired by backfilling burrows with cement grout and/or excavating and replacing burrowed materials with engineered fill.

Encroachments

As summarized and described in Table 2, encroachments identified during our field reconnaissance include buried utilities, irrigation lines, and trees located on, beneath, or adjacent to levee and stream bank slopes. While most of the observed encroachments do not appear to significantly affect the potential for seepage or embankment and foundation stability, some may become sources of concern if not properly managed and/or maintained.

Encroachments observed during our field reconnaissance that are of particular interest include the following:

- Utility crossings, including existing sanitary sewer lines that cross at approximately Stations 27+10 and 39+50.
- Irrigation controls and piping on outboard slope
- Trees and shrubs growing on the outboard slope

Levee construction plans (Reimer, 1991) indicate the presence of an 18-inch diameter sanitary sewer pipe that runs beneath the levee near Station 27+10 and a 48-inch diameter RCP sanitary sewer pipe that runs beneath the levee near Station 39+50. As shown on the construction plans, the sanitary sewer pipeline inverts are located at approximately 20 feet and 25 feet below the levee crest for the 18-inch and 48-inch diameter pipes, respectively. Levee construction plans also show crossings of storm drain pipelines beneath the levee near Station 26+63. No evidence of the 48-inch diameter pipe (e.g., manholes, surface markers) was observed during our field reconnaissance.

Irrigation pipelines that are damaged or improperly operated may cause saturation, erosion, and/or softening of surrounding soil. If necessary, repairs to damaged utilities may require excavation and replacement of levee embankment and foundation materials. Backfill materials around buried pipelines may provide a conduit for seepage beneath the levee. If backfill materials are not properly compacted, long-term settlement of the backfill also may cause deformation of overlying levee foundation and fill materials.

Decaying roots from trees growing on the outboard slope may weaken the embankment and foundation materials. Also, the roots may act as conduits for seepage into the levee slopes. Fallen or uprooted trees may damage the levee and stream bank slopes, leading to potential erosion and slope instability.

Further studies should examine the condition of and need for removing, relocating, and/or maintaining existing encroachments along the Lower Penitencia Creek east bank levee. Ongoing levee maintenance programs should include efforts to monitor and manage encroachments as appropriate. These efforts may include pruning or removing unhealthy, unstable, and/or interfering trees from levee and stream bank slopes. All encroachments should be managed and maintained in a manner that limits the potential for seepage, erosion, and instability of levee embankments and foundations.

OTHER ISSUES OF POTENTIAL CONCERN

Some non-geotechnical issues of potential concern also were observed during site reconnaissance activities. As summarized and described in Table 2, these issues include the presence and condition of outfalls and other embankment penetrations, adverse channel conditions, and miscellaneous maintenance considerations. The following sections describe these issues and discuss observations that may be relevant to further study, investigation and/or analysis by others.

Outfalls and Other Embankment Penetrations

During our field reconnaissance, existing storm drain outfalls and other levee penetrations were identified and observed for evidence of blockage, damage, and/or distress. Where accessible, the presence and condition of backflow prevention devices also were observed and documented.

One storm drain outfall, located near Station 26+00, was observed during our field reconnaissance. At the time of the reconnaissance, the outfall, consisting of a 72-inch diameter, corrugated metal (CMP) pipe, was about two thirds submerged in the low flow channel. An underground access vault was observed on the outboard side of the levee, but was not accessible at the time of the reconnaissance. A point-of-discharge backflow prevention device, such as a check valve or flap gate, was not observed where the outfall discharges into the creek channel. However, construction plans for the outfall indicate the presence of a flap gate within the vault on the outboard side of the levee.

More detailed reconnaissance of storm drain manholes and vaults on the outboard sides of the levees may be considered as part of further studies. To maintain the effectiveness of the levees and associated flood control systems, ongoing maintenance programs should include regular inspection of outfalls, penetrations, and closures. Outfalls and closures should be cleared of debris and maintained in good working order to avoid backflow during periods of high water inside the levees.

Adverse Channel Conditions

During our field reconnaissance, vegetation, debris, and other channel obstructions were observed in some areas. Relatively thick deposits of sediment and dense vegetation were observed in the bottom of the concrete channel near the Milmont Avenue Bridge. The sediment deposits appear to be particularly thick along the east side of the channel. Sediment deposits in this area may obstruct weep holes in the lining, which may cause excess pore pressures to develop behind the concrete.

Hydraulic analyses performed for levee recertification should consider these potential channel conditions during development of estimated flood elevations. If necessary, vegetation and debris should be cleared from the channel to increase channel capacity and maintain an acceptable amount of freeboard above design flood elevations.

The stream channel should be regularly inspected for blockages and other adverse conditions that may lead to increased flood elevations and/or erosion of stream bank and levee slopes. As part of regular maintenance work, excess sediment and debris should be removed from the channel as necessary to maintain adequate flow. Heavy vegetation and other potential debris-trapping obstructions also should be identified and cleared from the channel if necessary.

Miscellaneous Maintenance Considerations

Miscellaneous maintenance considerations identified during our field reconnaissance are generally related to control of vegetation and burrowing animals. These considerations are discussed in relevant sections above and summarized and described in Table 2.

CONCLUSION

In our opinion, levee and channel conditions observed along the Lower Penitencia Creek east bank levee do not generally indicate serious geotechnical problems that could jeopardize potential levee recertification. However, a number of minor geotechnical issues and areas of potential concern were noted during the reconnaissance. Further investigation, study, and analysis of these issues are necessary to complete the recertification process and demonstrate adequate performance of the existing levees under FEMA-designated loading conditions. It is anticipated that most issues of potential concern identified during recent field reconnaissance activities can be addressed through appropriate levee and channel maintenance programs.

REFERENCES

Reimer Associates, 1991, Improvements of the Easterly Levee of Lower Penitencia Creek, Milpitas, California.

TABLES

TABLE 1

SUMMARY OF REACHES - LEVEE FIELD RECONNAISSANCE

Lower Penitencia Creek East Bank Levee Recertification Project, Phase C2
Milpitas, California

		Approximate Station		Approximate Levee Dimensions					Features of Interest				
Reach	Location/Description	From	To	Length (ft)	Height above Outboard Toe (ft)	Crest Width (ft)	Inboard Slope (degrees)[1]	Outboard Slope (degrees)[1]	Erosion Protection	Penetrations	Flood Wall	Other Structures	Utilities
1	Downstream end of levee, from the California Circle bridge to a District gate that crosses the levee crest, approximately 740 feet south of the bridge, with a paved bicycle/pedestrian trail along the levee crest and an unimproved maintenance road along the inboard toe of the graded levee/stream bank slope. Near the northwest end of the reach, the creek flows through an open, concrete-lined channel with a trapezoidal section that directs water westward beneath and beyond the California Circle bridge.	11+40	18+80	740	3 - 7	11 - 12	27 - 30 (soil slopes) 27 - 45 (concrete slopes)	20 - 26 (soil slopes)	X			X	X
2	From approximately 740 feet southeast of the California Circle bridge to approximately 240 feet north of the Milmont Drive bridge, with a lightly gravel-surfaced maintenance road along the levee crest and an unimproved maintenance road along the inboard toe of the graded levee/stream bank slope.	18+80	37+60	1,880	4 - 10	10 - 12	25 - 28 (soil slopes) 27 - 35 (concrete slopes)	<10 - 30 (soil slopes) 27 - 45 (concrete slopes)	X	X		X	X
3	From approximately 240 feet north of the Milmont Drive bridge to the confluence of Penitencia and Berryessa Creeks, about 500 feet south of the Milmont Drive Bridge, with a paved and gravel-surfaced maintenance road along the levee crest. The creek flows through an open, concrete-lined channel with a trapezoidal section.	37+60	2+00[2]	815	4 - 8	10 - 20	25 - 30 (soil slopes) 27 - 45 (concrete slopes)	10 - 25 (soil slopes)	X			X	X

Notes:

1. Slopes were estimated in the field using a Brunton compass; values are presented separately for soil and concrete-surfaced slopes.
2. Reach 3 extends from approximately Station 37+60 on the Penitencia Creek control line to approximately Station 2+00 on the Berryessa Creek control line. The Berryessa Creek control line begins at approximately Station 43+75, where it diverges from the Penitencia Creek line (i.e., Station 0+00 on the Berryessa Creek line is located at approximately Station 43+75 on the Penitencia Creek line).

TABLE 2

SUMMARY OF KEY OBSERVATIONS - LEVEE FIELD RECONNAISSANCE

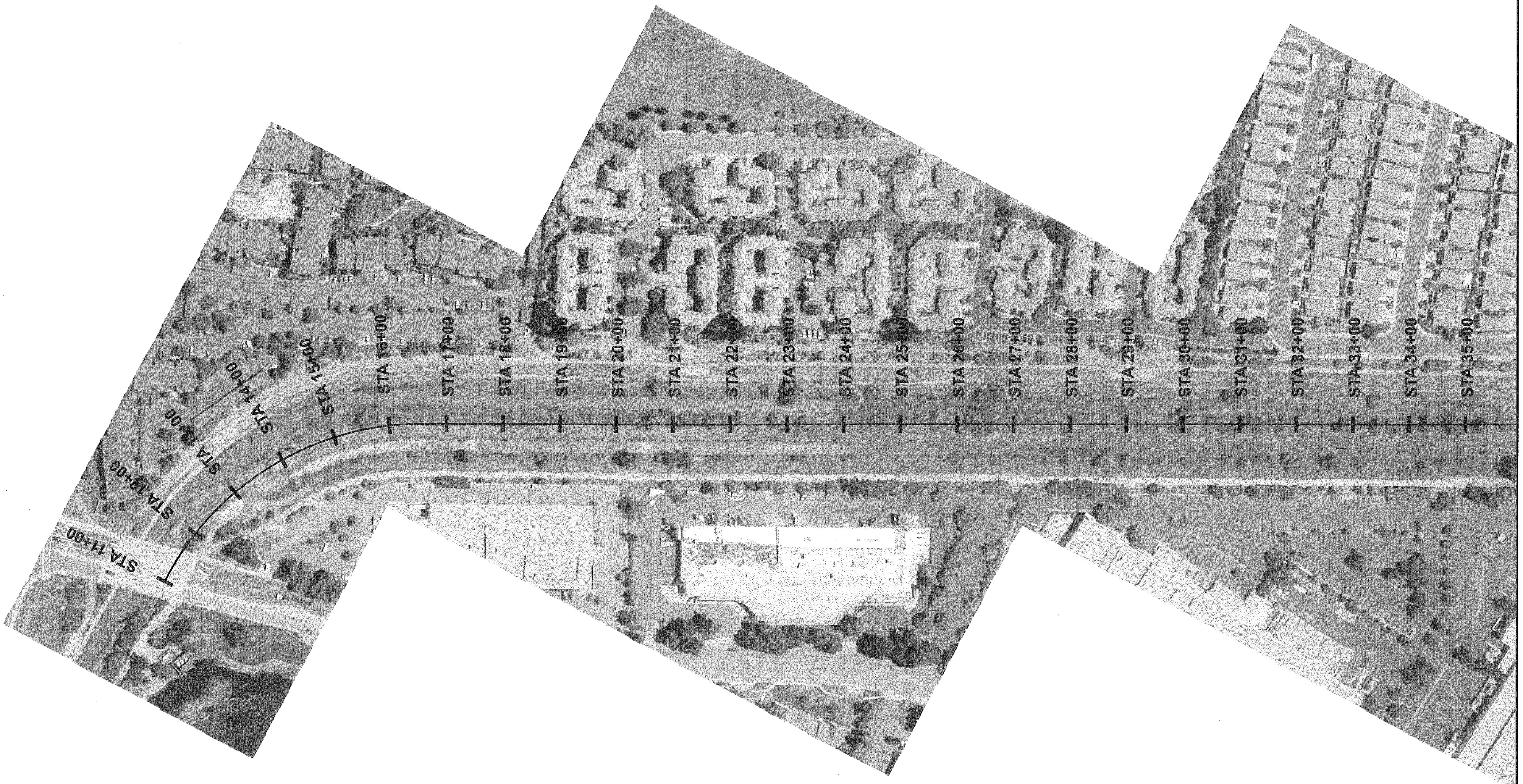
Lower Penitencia Creek East Bank Levee Recertification Project, Phase C2
Milpitas, California

Reach	Station		Key Observations	Observed Condition and Degree of Severity^1											Potential Geotech Issue?	Follow-up	By^2	Comments/Notes
	From	To		Settlement	Erosion	Stability	Ground Cracks or Deformation	Encroachments	Maintenance	Structural Problems	Seepage	Closures or Drainage	Adverse Channel	Animal Burrows				
1	11+40	18+80	Erosion by raveling of aggregate base material on the inboard and outboard sides of the levee crest; in some areas, the edge of the asphalt-concrete pavement is undermined and cracked		1											maintenance	SCVWD	
			Moderately steep (~27-30°) inboard slopes			1									X	exploration/analysis	PGE/AMEC	
			Traverse cracks in asphalt-concrete pavement at crest, generally coincident with trees and large shrubs on the outboard levee slope				2									maintenance	SCVWD	
			Landscaping vegetation (small trees and shrubs) on outboard slope					2								maintenance	SCVWD	See Note 5
			Irrigation controls and piping on outboard slope					1								none	–	
			Standing water and wet soil at outboard toe (12+80)						3		1					maintenance	SCVWD	
			Minor burrows on inboard levee and stream bank slopes, mostly gopher^4											1	X	maintenance	SCVWD	
2	18+80	37+60	Erosion gully and/or narrow scarp and slide feature on inboard levee slope (28+00)		2	3									X	exploration/analysis	PGE/AMEC	
			Moderately steep (~25-28°) inboard and locally steep (~30°) outboard soil slopes			1									X	exploration/analysis	PGE/AMEC	
			Moderately steep to steep (~27-35°) concrete-surfaced inboard slopes and moderately steep to very steep (~27-45°) concrete-surfaced outboard slopes (20+40 to 22+30, 28+20 to 30+00, and 36+10 to 38+00); with the exception of outboard slopes above retaining walls, no provisions for relief of excess pore pressure behind the concrete-surfaced slopes (i.e., drains or weep holes) were observed			2									X	exploration/analysis	PGE/AMEC	
			Longitudinal crack (up to approx. 1" wide) on the lower half of the inboard levee slope, approximately 5 feet long, filled with soil at the surface but shows approximately 3 inches of vertical offset (30+50)			3									X	exploration/analysis	PGE/AMEC	
			Lower portion of the levee/stream bank slope and the lower maintenance road are rutted and hummocky (19+30, 26+30)			2	3								X	exploration/analysis	PGE/AMEC	
			Landscaping vegetation (small trees and shrubs) on outboard slope					2								maintenance	SCVWD	See Note 5
			Irrigation controls and piping on outboard slope					1								none	–	
			Storm drain outfall; unable to view backflow prevention device (26+00)									1				inspection/maintenance	SCVWD	
			Minor burrows on inboard and outboard levee and stream bank slopes, mostly gopher^4											1	X	maintenance	SCVWD	
3	37+60	2+00^3	Inboard soil slope between the top of the concrete channel and the levee crest is bare with some raveling of granular material and early rill development (41+00 to 0+80)		3											maintenance	SCVWD	
			Moderately steep (~25-30°) inboard soil slopes between the top of the concrete channel and the levee crest			2									X	exploration/analysis	PGE/AMEC	
			Asphalt-concrete pavement is heavily cracked and damaged in some areas (42+20 to 0+90)				3									maintenance	SCVWD	
			Landscaping vegetation (small trees and shrubs) on outboard slope					1								maintenance	SCVWD	See Note 5
			Irrigation controls and piping on outboard slope					1								none	–	
			Drain holes near mid-height of the concrete channel wall (0+50 to 1+50) appear to be clogged with vegetation, drain holes near the base of the concrete channel wall (38+00 to 1+50) could not be observed but appear to be buried by sediment			2			3	2					X	analysis/maintenance	AMEC/SCVWD	
			Relatively thick sediment deposits and dense reeds in the bottom of the concrete channel, particularly against the right (east) channel slope in the vicinity of the Milmont Avenue bridge										3			analysis/maintenance	S&W/SCVWD	See Note 5

Notes:

1. Severity Code: 1 = Slight, 2 = Slight to Moderate, 3 = Moderate, 4 = Moderate to Severe, 5 = Severe
2. AMEC = AMEC Geomatrix, PGE = Pacific Geotechnical Engineering, S&W = Schaaf & Wheeler, SCVWD = Santa Clara Valley Water District
3. Reach 3 extends from approximately Station 37+60 on the Penitencia Creek control line to approximately Station 2+00 on the Berryessa Creek control line. The Berryessa Creek control line begins at approximately Station 43+75, where it diverges from the Penitencia Creek line (i.e., Station 0+00 on the Berryessa Creek line is located at approximately Station 43+75 on the Penitencia Creek line).
4. For classification purposes, holes less than approximately 2 to 3 inches in diameter are considered indicative of gopher burrows and holes greater than approximately 2 to 3 inches in diameter are considered indicative of ground squirrel burrows
5. Others should evaluate the actual need for corrective action and develop appropriate corrective measures, if necessary

FIGURES

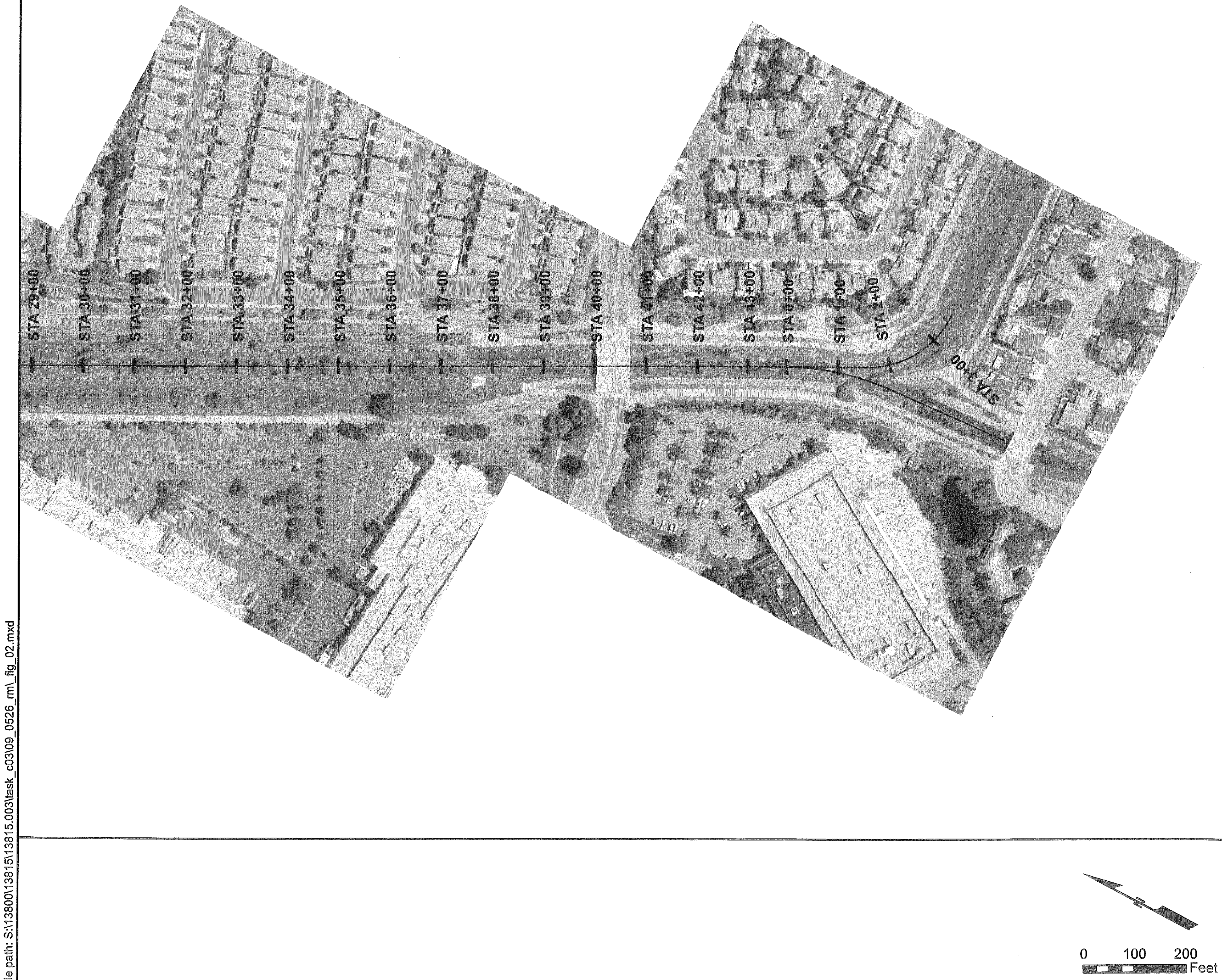


EAST BANK LEVEE
Lower Penitencia Creek Levee
FEMA Levee Certification
Milpitas, California

By: DMO | Date: 5/26/2009 | Project No. 13815.003

AMEC Geomatrix | Figure 1

File path: S:\13800\13815\13815.003\task_c03109_0526_rm_fig_01.mxd



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EAST BANK LEVEE
Lower Penitencia Creek Levee
FEMA Levee Certification
Milpitas, California

By: DMO	Date: 5/26/2009	Project No. 13815.003
AMEC Geomatrix		Figure 2

ATTACHMENT 1

FIELD RECONNAISSANCE LOGS

GEOMATRIX

LEVEE FIELD RECONNAISSANCE LOG

PAGE 1 OF 4

Project Name: Lower Penitencia Creek Levee Recertification
Date: 8/12/08
Project Number: 13815.003.0
Weather Conditions: Sunny, Warm ≈ 80°
Location: Milpitas, CA
Observations Recorded by: D. Ritzman

Station Interval
From: 11+40 To: 18+80

Other Reach/Location ID
Begin at California Circle Bridge

LEVEE DESCRIPTION

Geometry and Configuration:

Means of Measurement/Estimation

Height above Outboard Toe:	3-7 ft	☐ Visual	☒ Rod and Hand Level		☐ Other	
Crest Width:	11-12 ft	☐ Visual	☐ Pace	☒ Tape	☐ Other	
* Inboard Slope:	27-30 [✓ deg] H:V %	☐ Visual	☒ Brunton Compass		☐ Other	
Outboard Slope:	20-26 [✓ deg] H:V %	☐ Visual	☒ Brunton Compass		☐ Other	

☒ Erosion Protection	Type: Concrete Channel Slope	Location: 11+40 - 14+10	Condition: ok
☐ Penetrations	Type:	Location:	Condition:
☐ Floodwall	Type:	Location:	Condition:
☒ Other Structures	Type: California Circle Bridge	Location: 10+60 - 11+40	Condition:
☒ Utilities	Irrigation Lines along outboard slope	11+40 - 18+80	

Ground Cover:

	Inboard Slope	Outboard Slope	Crest	Comments
Bare Ground	☐	☒	☐	
Grass	☒	☐	☐	
Shrubs	☐	☒	☐	Outboard slope is landscaped
Trees	☐	☒	☐	w/ shrubs and ~~the~~ small trees.
Riprap	☐	☐	☐	
Gravel	☐	☐	☐	
Pavement	☒ Concrete	☐	☒ A/C	Bicycle Trail (paved) along crest
Other	☐	☐	☐	

Additional Explanation/Notes:

Levee reach follows the outside of a bend in the creek channel between the California Circle Bridge (≈ Sta. 11+40) and ~~[illegible]~~ a SCVWD gate across the crest of the levee (≈ Sta. 18+80).

Channel slopes are Concrete between ≈ Sta. 11+40 and 14+10. Concrete slopes vary from about 2:1 (27°) to 1:1 (45°). An access road runs along the inboard toe of the levee/streambank slope, between the levee

☒ Continued at end of report

SURFACE WATER CONDITIONS

Flow conditions and water levels on the inboard side of the levee:

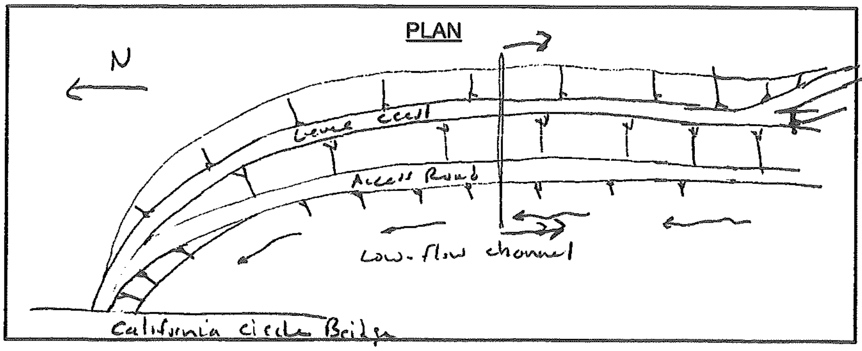
Incoming Tide in low-flow channel

☐ Continued at end of report

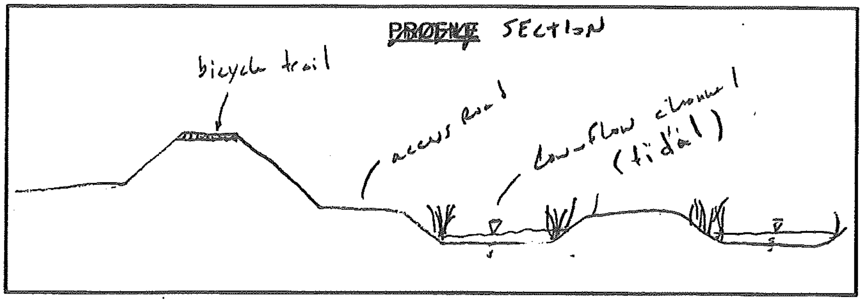
PROJECT NUMBER: 13815.003.0 **PAGE** 2 **OF** 4

Station Interval	**Other Reach/Location ID**
From: 11+40 To: 18+80	

SKETCHES



NOTES



SECTION

OBSERVATIONS

Check and indicate severity of all that apply:

		Slight		Moderate	Severe
☐	**Evidence of settlement** (i.e., surface cracks, sinkholes, hummocky/undulating ground)	☐		☐	☐
1) ☒	**Erosion** (rills, gullies, undercutting of levee slopes, scour, etc.)	☒		☐	☐
4) ☒	**Evidence of potential slope stability problems** (as indicated by steep and/or undercut slopes, surface cracks, caving, bulging, slumping, or irregular ground surfaces, etc.)	☒		☐	☐
2) ☒	**Other ground deformation/cracking** (not obviously related to settlement, erosion, or instability)	☐	X	☐	☐
3) ☒	**Encroachments by vegetation or other improvements**	☐	X	☐	☐
☐	**Inadequate maintenance** (i.e, mowing/vegetation control, inoperable control structures, etc.)	☐		☐	☐
☐	**Evidence of potential structural problems** (walls, abutments, weirs, aprons, etc.)	☐		☐	☐
4) ☒	**Evidence of seepage** (e.g., wetland-type vegetation, moist or wet ground, springs, boils, etc.)	☐	X	☐	☐
☐	**Damaged, distressed, and/or blocked closures or interior drainage structures** (including gates, flaps, and/or other devices for backflow control)	☐		☐	☐
☐	**Adverse channel conditions** (i.e., blockages, excessive debris, sand/gravel bars, etc.)	☐		☐	☐
5) ☒	**Animal burrows**	☒		☐	☐

Further describe the location, extent and nature of checked items:

1) Minor erosion along edge of pavement section at crest of inboard and outboard slopes. In some areas, pavement is ~~[illegible]~~ breaking off along the edge because it has been undermined by underlying AB having raveled down slope. Erosion may be caused in part by foot traffic on outboard slope used for levee access.

2) Pavement cracks, most perpendicular to slope, appear to be caused by ~~[illegible]~~ shallow tree/shrub roots beneath the pavement. ~~[illegible]~~ Section of pavement near Sta. 16+50 appears to have been recently replaced for this reason.

☒ Continued at end of report

DESCRIPTIONS AND OBSERVATIONS, CONTINUED

Description, continued:

embankment and the low-flow channel. The access road ~~is~~ is unimproved dirt w/ occasional thin gravel surfacing - does not appear to be all-weather.

From Sta. 14+10 to 18+80, inboard ~~[illegible]~~ levee / streambank slope is vegetated w/ grass and weeds.

Outboard slope is ~~[illegible]~~ landscaped with ~~&~~ shrubs and small trees and scattered ground cover. An irrigation system on the outboard slope serves the ~~[illegible]~~ landscaped areas.

A paved ~~[illegible]~~ bicycle/pedestrian trail ~~[illegible]~~ runs along the crest of the levee.

Observations, Continued:

3) Landscaping (trees/shrubs) on outboard slope w/ irrigation system.

5) ~~[illegible]~~ Minor gopher burrows (none appear to be active) on inboard slope.

4) Small patch of wet soil and standing ~~[illegible]~~ water at outboard toe, Sta. 12+80. Water level on inboard side of levee is much lower than elevation at outboard toe. Suspect ~~[illegible]~~ the moisture may be due to an irrigation system leak or other transient source.

6) Moderately steep inboard slopes

PROJECT NUMBER: 13815.003.0 **PAGE** 4 **OF** 4

Station Interval		Other Reach/Location ID
From: 11+40	To: 18+80	

PHOTOS

Photo ID	Location	Description
1	11+40	E. View from California Circle Bridge
2-7	12+80	E/W Views of outboard slope, crest, and inboard slope
8	12+80	Standing water @ outboard toe (irrigation leak?)
9, 10	12+80	Undermined pavement / erosion at levee access.
11	13+10	Pavement cracks (suspect roots)
12-17	14+80	E/W Views of outboard slope, crest, and inboard slope
18	15+60	Outboard slope - broken edge of pavement, irrigation system
19, 20	16+10	Pavement cracks (roots)
21 - 24	17+00	S/N Views of outboard / inboard slopes

SURVEY POINTS

Point ID	Location	Description

GEOMATRIX

LEVEE FIELD RECONNAISSANCE LOG

PAGE 1 OF 6

Project Name: Lower Penitencia Creek Levee Recertification

Date: 8/12/08

Project Number: 13815.003.0

Weather Conditions: Sunny, Warm, ~80°

Location: Milpitas, CA

Observations Recorded by: D. Ritzman

Station Interval		Other Reach/Location ID
From: 18+80	To: 37+60	

LEVEE DESCRIPTION

Geometry and Configuration:

		Means of Measurement/Estimation			
Height above Outboard Toe:	4 - 10 ft	☐ Visual	☒ Rod and Hand Level		☐ Other
Crest Width:	10-12 ft	☐ Visual	☐ Pace	☒ Tape	☐ Other
Inboard Slope:	25-28 [✓ deg] H:V %	☐ Visual	☒ Brunton Compass		☐ Other
Outboard Slope:	<10 - 30 [✓ deg] H:V %	☐ Visual	☒ Brunton Compass		☐ Other

☒ Erosion Protection	Type: Concrete facing on inboard slope	Location:	Condition: ok ✓
☒ Penetrations	Type: S.D. outfall (72"), S.S. Line (18")	Location:	Condition: ok ✓
☐ Floodwall	Type:	Location:	Condition:
☒ Other Structures	Type: Retaining Walls on outboard slope	Location:	Condition: ok ✓
☒ Utilities (See additional explanation)			

Ground Cover:

	Inboard Slope	Outboard Slope	Crest	Comments
Bare Ground	☐	☒	☒	outboard slope is landscaped
Grass	☒	☐	☐	with trees and shrubs
Shrubs	☐	☒	☐	(irrigated)
Trees	☐	☒	☐	
Riprap	☐	☐	☐	Dirt/gravel path on levee crest
Gravel	☐	☐	☒	
Pavement	☒	☒	☐	
Other	☐	☐	☐	

Concrete aprons for slope stability and protection at electrical tower locations

Additional Explanation/Notes:

Levee along this reach is relatively uniform with the exception of 3 locations where high voltage electrical towers encroach on the outboard levee slope. At these 3 locations, the crest jogs inboard and the inboard and outboard slopes have been steepened to create space for an access road / bicycle / pedestrian path between the towers and the outboard toe of the levee slope. In most locations the outboard levee slope is about 4 to 7 feet high. However, the electrical towers sit in closed depressions that are several feet lower than the surrounding ground surface.

☒ Continued at end of report

SURFACE WATER CONDITIONS

Flow conditions and water levels on the inboard side of the levee:

Tidal flow (slack - outgoing) in low-flow channel

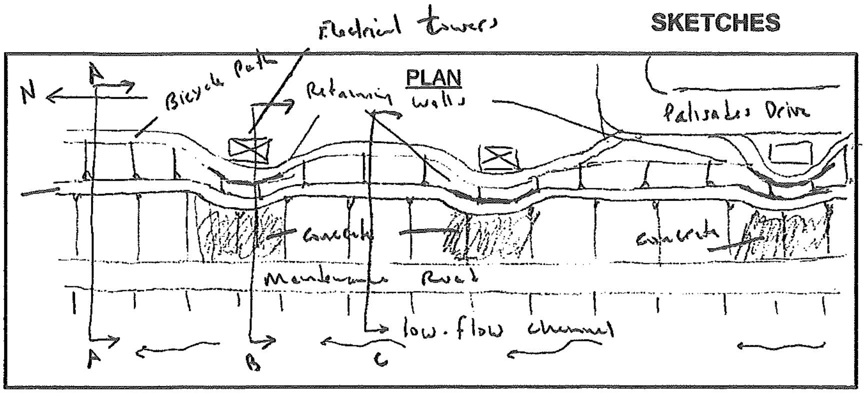
☐ Continued at end of report

PROJECT NUMBER: 13815.003.0 **PAGE** 2 **OF** 6

Station Interval	Other Reach/Location ID
From: 18+80 To: 37+60	

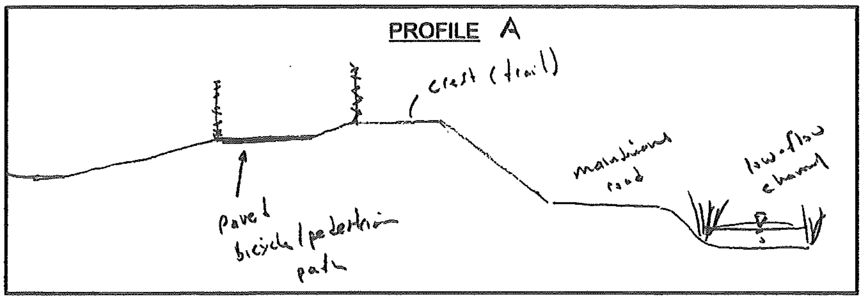
SKETCHES

levee crest



NOTES

See attached sheet for profiles B and C



SECTION

OBSERVATIONS

Check and indicate severity of all that apply:

			Slight		Moderate	Severe
	☐	**Evidence of settlement** (i.e., surface cracks, sinkholes, hummocky/undulating ground)	☐		☐	☐
5)	☒	**Erosion** (rills, gullies, undercutting of levee slopes, scour, etc.)	☐	X	☐	☐
5), 6)	☒	**Evidence of potential slope stability problems** (as indicated by steep and/or undercut slopes, surface cracks, caving, bulging, slumping, or irregular ground surfaces, etc.)	☐		☒	☐
1)	☒	**Other ground deformation/cracking** (not obviously related to settlement, erosion, or instability)	☐		☒	☐
2)	☒	**Encroachments by vegetation or other improvements**	☐	X	☐	☐
	☐	**Inadequate maintenance** (i.e, mowing/vegetation control, inoperable control structures, etc.)	☐		☐	☐
	☐	**Evidence of potential structural problems** (walls, abutments, weirs, aprons, etc.)	☐		☐	☐
	☐	**Evidence of seepage** (e.g., wetland-type vegetation, moist or wet ground, springs, boils, etc.)	☐		☐	☐
3)	☒	**Damaged, distressed, and/or blocked closures or interior drainage structures** (including gates, flaps, and/or other devices for backflow control)	☒		☐	☐
	☐	**Adverse channel conditions** (i.e., blockages, excessive debris, sand/gravel bars, etc.)	☐		☐	☐
4)	☒	**Animal burrows**	☒		☐	☐

Further describe the location, extent and nature of checked items:

1) Lower maintenance road is rutted and hummocky in some areas. Appears to be a result of vehicle traffic and perhaps localized excavation/fill to create ~~by~~ bicycle ~~path~~ ~~jumps~~ jumps (Sta. 19+30, 26+30). Some oversteepening of levee toe at 26+30.

2) Landscaping (trees/shrubs) on outboard slope (with irrigation system)

3) No backflow prevention device observed for 72" CMP outfall

4) Minor burrow activity (gopher) noted on inboard and outboard slopes

5) Minor erosion gully (?) developing on inboard slope at ≈ Sta. 28+00

☒ Continued at end of report

18+80 to ~~[illegible]~~ 37+60

PROJECT NUMBER: 13815.003.0 **PAGE** 3 **OF** 6

DESCRIPTIONS AND OBSERVATIONS, CONTINUED

Description, Continued:

As a result, the outboard levee slope is typically about 7 to 9 feet high in the areas around the electrical towers.

- The levee crest is about 10 to 12 feet wide and is unpaved, bare ground with a minor amount of gravel embedded in the soil at the surface.
- The inboard levee/stream bank slope is graded and relatively uniform, between about 25 and 28 degrees. The inboard slope is typically ~ 12 ft. (+/-) high. An unimproved (bare ground) maintenance road runs along the toe of the inboard levee/streambank slope. The maintenance road doesn't appear to be all-weather. The low-flow/tidal channel runs along the inboard side of the maintenance road. For about 50 ~~[illegible]~~ feet upstream and downstream of each electrical tower, the inboard slope becomes steeper to accommodate an inboard jog in the crest. The steepened slopes are covered in rough-textured concrete and have maximum slopes of about 35 degrees.
- The outboard slopes are variable, with slopes ranging from <10 degrees to about 30 degrees. The outboard slopes are landscaped with small trees and shrubs, which are supported by an irrigation system. A paved bicycle/pedestrian access road runs along the outboard side of the levee between Stations 18+80 and 32+00 and between Stations 36+00 and 40+00. Between Stations 32+00 and 36+00 a sidewalk ~~adj~~ adjacent to Palisades Drive runs along the toe of the levee. The paved trail typically runs along the outboard toe of the levee, although between Stations 18+80 and 26+00 it appears ~~[illegible]~~ be on the very shallow outboard slope. At electrical tower locations, the paved trail runs between the outboard toe of the levee and the electrical tower. For about 100 feet upstream and downstream of each electrical tower, the outboard slope becomes steeper to ~~[illegible]~~ create space for the trail between the tower and levee. The steepened slopes are covered in rough-textured concrete. For about 50 feet upstream and downstream of each tower, the bottom of the outboard slope is supported by a concrete retaining wall. Wall heights range from about ~~[illegible]~~ 0.5 to 4.5 feet. From the top of the vertical wall, the concrete-faced slope continues upward to the crest of the levee. Concrete-faced slopes upstream and downstream of the wall and above the wall range from about 2:1 (27°) to 1:1 (45°).
- Electrical Towers are located at approximately Stations 21+30, 29+10, and 37+00.

DESCRIPTIONS AND OBSERVATIONS, CONTINUED

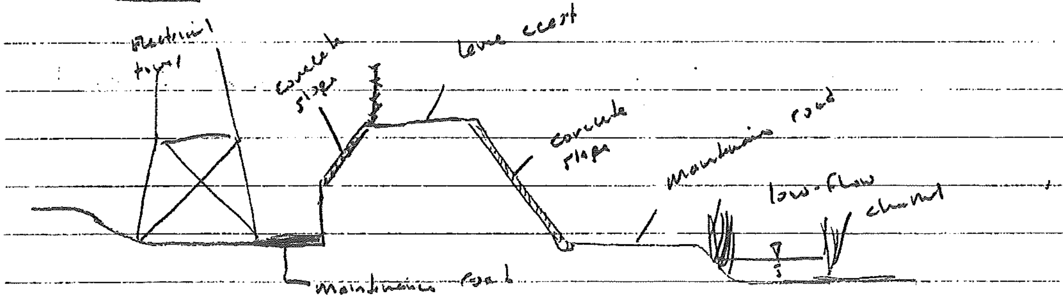
Description, continued:

Concrete-faced slopes and retaining walls are located as follows:

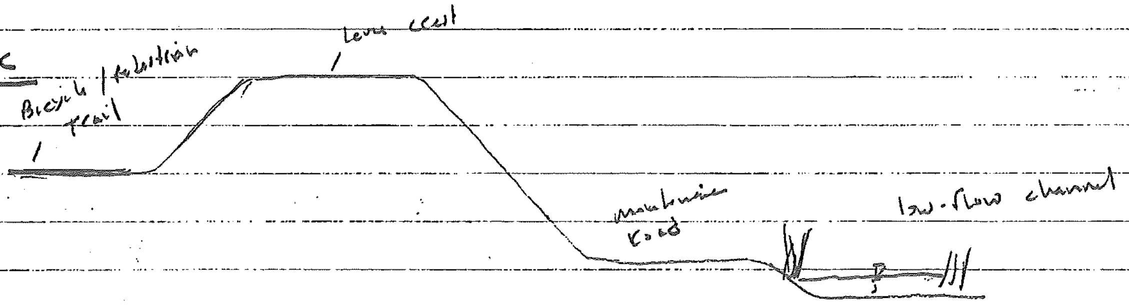
Concrete Slope Outboard Side	Retaining Wall Outboard Side	Concrete Slope Inboard Side
20+40 to 22+30	20+90 to 21+80	20+90 to 21+80
28+20 to 30+00	28+70 to 29+70	28+70 to 29+70
36+10 to 38+00	36+60 to 37+60	36+60 to 37+60

A 72-inch diameter CMP outfall is also located at approximately Station 26+00. There is no flap gate on the outfall, which was about 2/3 submerged below tidal levels in the low-flow channel at the time of the reconnaissance. An underground vault was observed on the outboard side of the levee, opposite the outfall. Other utilities observed near the levee include a sanitary sewer line that runs beneath the levee near Sta. 26+10, where a manhole was observed on the outboard side. Available construction plans show the 18" diam. sewer line running at an angle beneath the levee and creek. However, no manholes were observed on the opposite side of the creek. An irrigation system is located along the outboard levee slope.

Profile B



Profile C



OBSERVATIONS, Continued:

6) Ground crack observed in lower inboard levee slope @ Sta. 30+50. Crack is ≈ 5 ft long and filled, but it looks like there is about 2-3" of vertical offset and lower portion of slope is somewhat hummocky in this area.

PROJECT NUMBER: 13815.003.0 **PAGE** 5 **OF** 6

Station Interval

From: 18+80 **To:** 37+60

Other Reach/Location ID

PHOTOS

Photo ID	Location	Description
25, 26	26+00	72" CMP Storm Drain outlet (≈ 2/3 submerged)
27 – 30	18+80	~~W~~ S/N views of outboard / inboard slopes
31, 32	20+00	S/N view of inboard slope and crest
33, 34	20+80	S view inboard ~~[illegible]~~ concrete slopes
35	21+00	~~S~~ ~~[illegible]~~ outboard ret. wall and tower
36, 37	21+80	N. view inboard / outboard concrete slopes
38, 39	24+00	S/N view of inboard slope and crest
40, 41	26+00	S/N views of inboard slope (from maintenance road)
42, 43	28+50	S views of inboard concrete slope and crest
44	28+70	outboard retaining wall and tower (S. view)
45	29+60	" " " " " (N. view)
46	29+90	N view of inboard concrete slope and crest
47–48	33+00	S/N views of inboard slope and crest
49	36+20	S view of concrete slope and crest
50–51	36+20	S/N views of inboard slope (from maintenance road)
52	36+60	Outboard retaining wall and tower (S. view)
53	~~[illegible]~~ 37+60	" " " " " (N. view)
54, 55	37+60	S/N views of inboard slope and levee crest
56, 57	37+60	S/N views of outboard slope + ret. wall
58	37+50	Retaining wall / concrete outboard slope

SURVEY POINTS

Point ID	Location	Description
1	26+00	72" SD outlet, Receiver held ~ ~~12"~~ 12 ft above
		invert of CMP pipe (6 ft. above top of pipe)
2	26+00	72" SD vault on outboard side of levee (unit set on top of vau
3	26+10	SS manhole on " " " " (unit set on top)
59, 60	36+60	Ret. wall and concrete slope (outboard)
61, 62	35+00	S/N views of outboard slope
63, 64	33+00	S/N views of outboard slope
65	31+90	N view of outboard slope
66, 67	30+20	S/N view of outboard slope (Conc. slope + wall)
68	29+80	Concrete wall and slope
69	28+70	" " " "
70, 71	28+20	S/N view of outboard slope
72, 73	26+50	" " " " "
74, 75	26+00	SD vault and SS manhole
76	25+70	S. view, SD vault and SS manhole
77, 78, 79	25+00	S/N views of outboard slope
80, 81	23+50	S/N views of outboard slope
82, 83	22+50	" " " "

PROJECT NUMBER: 13815.003.0 **PAGE** 6 **OF** 6

Station Interval		Other Reach/Location ID
From: 18+80	To: 37+60	

PHOTOS

Photo ID	Location	Description
84, 85	21+00	Concrete Wall and slope
86, 87	20+90	" " " "
88, 89	19+50	S/N views of outboard slope
90, 91, 92	19+30	Ruts and "bump" in ~~access~~ lower maintenance road
93, 94	26+30	Rutting / oversteepening at levee toe.
95, 96	28+00	Minor erosion gully on inboard slope
97	29+50	Rutting at toe of inboard slope
98, 99, 100	30+50	Ground crack on lower inboard slope

SURVEY POINTS

Point ID	Location	Description

GEOMATRIX

LEVEE FIELD RECONNAISSANCE LOG

PAGE 1 OF 4

Project Name: Lower Penitencia Creek Levee Recertification

Date: 8/13/08

Project Number: 13815.003.0

Weather Conditions: Sunny, warm (~80°)

Location: Milpitas, CA

Observations Recorded by: D. Ritzman

Station Interval

From: 37+60 To: 2+00

Other Reach/Location ID

South End of Study Area

LEVEE DESCRIPTION

Geometry and Configuration:

Means of Measurement/Estimation

Height above Outboard Toe:	4 - 8 ft	☒ Visual	☐ Rod and Hand Level		☐ Other
Crest Width:	10 - 20 ft	☐ Visual	☐ Pace	☒ Tape	☐ Other
Inboard Slope:	40 - 45 [✓ deg] H:V %	☐ Visual	☒ Brunton Compass		☐ Other
Outboard Slope:	10 - 25 [✓ deg] H:V %	☐ Visual	☒ Brunton Compass		☐ Other

☒ Erosion Protection	Type: Trapezoidal Concrete channel	Location: 37+60 to 2+00	Condition: (clogged weep holes)
☐ Penetrations	Type:	Location:	Condition:
☐ Floodwall	Type:	Location:	Condition:
☒ Other Structures	Type: Milmont Ave Bridge	Location: 40+00 to 40+80	Condition:

☑ Utilities electrical tower (1+20), Sanitary Sewer near 39+50 not observed, S.D. line near 41+00 also not observed.

Ground Cover:

	Inboard Slope	Outboard Slope	Crest	Comments
Bare Ground	☒	☒	☒	
Grass	☒	☒ } Landscaping	☐	
Shrubs	☐	☒	☐	
Trees	☐	☒	☐	
Riprap	☐	☐	☐	
Gravel	☒	☐	☒	
Pavement	☒ Concrete channel	☐	☒ - Concrete and A/C	
Other	☐	☐	☐	

Additional Explanation/Notes:

Levee in this reach is relatively complex. Lower portion of the inboard slope is a trapezoidal concrete channel section with approximately 1:1 (45°) side slopes. At the top of the concrete slope, a graded slope continues to the levee crest. The graded slope ranges from about 3 to 10 feet high and has a slope of about 25 to 30 degrees. The levee crest is bare ground between Stations 37+60 and 39+00, concrete from about 39+00 to 42+20 (Milmont Avenue Bridge excluded), A/C pavement from about 42+20 to 0+90, and gravel from about 0+90 to 2+00 (end of reach/study area).

☒ Continued at end of report

SURFACE WATER CONDITIONS

Flow conditions and water levels on the inboard side of the levee:

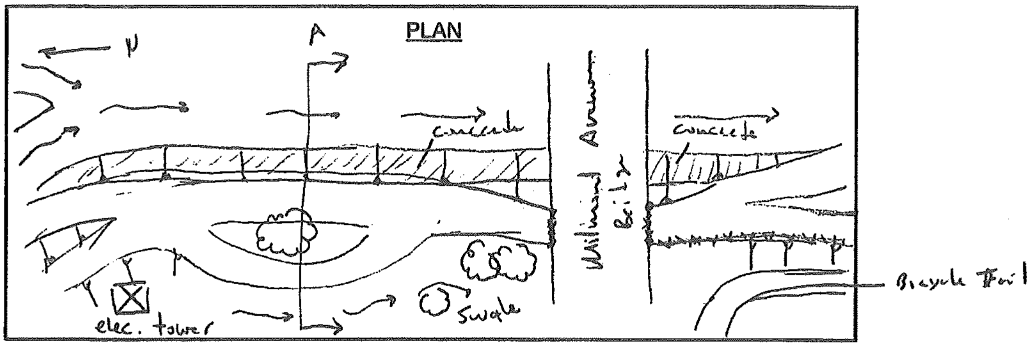
Minor flow in bottom of channel

☐ Continued at end of report

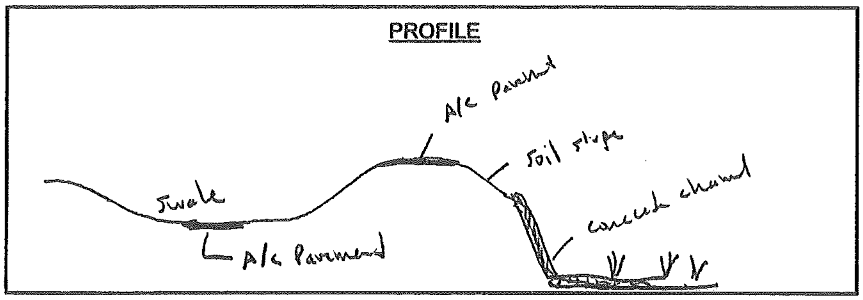
PROJECT NUMBER: 13815.003.0 **PAGE** 2 **OF** 4

Station Interval	Other Reach/Location ID
From: 37+60 To: 2+00	

SKETCHES



NOTES



SECTION

OBSERVATIONS

Check and indicate severity of all that apply:

		Slight		Moderate	Severe
	☐ **Evidence of settlement** (i.e., surface cracks, sinkholes, hummocky/undulating ground)	☐		☐	☐
1)	☒ **Erosion** (rills, gullies, undercutting of levee slopes, scour, etc.)	☐		☒	☐
	☐ **Evidence of potential slope stability problems** (as indicated by steep and/or undercut slopes, surface cracks, caving, bulging, slumping, or irregular ground surfaces, etc.)	☐	×	☐	☐
2)	☒ **Other ground deformation/cracking** (not obviously related to settlement, erosion, or instability)	☐		☒	☐
3)	☒ **Encroachments by vegetation or other improvements**	☒		☐	☐
4)	☒ **Inadequate maintenance** (i.e, mowing/vegetation control, inoperable control structures, etc.)	☐		☒	☐
4)	☒ **Evidence of potential structural problems** (walls, abutments, weirs, aprons, etc.)	☐	×	☐	☐
	☐ **Evidence of seepage** (e.g., wetland-type vegetation, moist or wet ground, springs, boils, etc.)	☐		☐	☐
	☐ **Damaged, distressed, and/or blocked closures or interior drainage structures** (including gates, flaps, and/or other devices for backflow control)	☐		☐	☐
5)	☒ **Adverse channel conditions** (i.e., blockages, excessive debris, sand/gravel bars, etc.)	☐		☒	☐
	☐ **Animal burrows**	☐		☐	☐

Further describe the location, extent and nature of checked items:

1) Inboard soil slope above concrete channel is bare with some ravelling and early rill development. (Sta. 41+00 to 0+80)

2) A/c Pavement between 42+20 and 0+90 is heavily cracked and damaged in some areas. Appears to be the result of age, wear, and heavy equipment loads rather than slope instability. May also be some expansive soil issues at edge of pavement.

3) Landscaping and irrigation systems on outboard slopes

4) Weep holes in concrete channel wall appear to be clogged. Holes at base of wall are buried in sediment. Potential concern for drainage behind the wall.

☒ Continued at end of report

37+60 to 2+00

DESCRIPTIONS AND OBSERVATIONS, CONTINUED

Description, continued:

- Inboard slope also includes concrete ramps from Sta. 37+60 to 38+80 and 0+90 to 2+20 that provide access to maintenance roads at the toes of inboard levee/stream bank slopes upstream and downstream of this reach.
- Outboard slope is landscaped w/ irrigation systems and small trees and shrubs. From 37+60 to about 38+00, outboard slope is concrete-faced (associated w/ electrical tower, as described for previous reach). Paved bicycle/pedestrian trail follows the outboard toe between about Sta. 37+60 to 40+00 (Milmont Avenue Bridge). South of the bridge, a grassy drainage swale runs along the outboard toe. Outboard slopes are variable, but typically shallow.

An electrical tower is located on the outboard side of the levee at about Station 1+20. Does not appear to affect levee configuration.

Construction Drawings show a 48" diameter sanitary sewer line that runs beneath the levee and creek just north of the bridge. However, manholes associated with the line were not observed near the levee. A storm drain line is also shown in the swale on the south side of the bridge, outboard side of levee, but no inlets or other evidence of the line was observed.

Observations, continued:

5) Heavy reed growth and sediment in bottom of channel. Sediment appears to block the line of drainage holes along the base of the concrete channel slope.

PROJECT NUMBER:	13815.003.0	**PAGE** 4	**OF** 4

Station Interval		**Other Reach/Location ID**
From: 37+60	**To:** 2+00	

PHOTOS

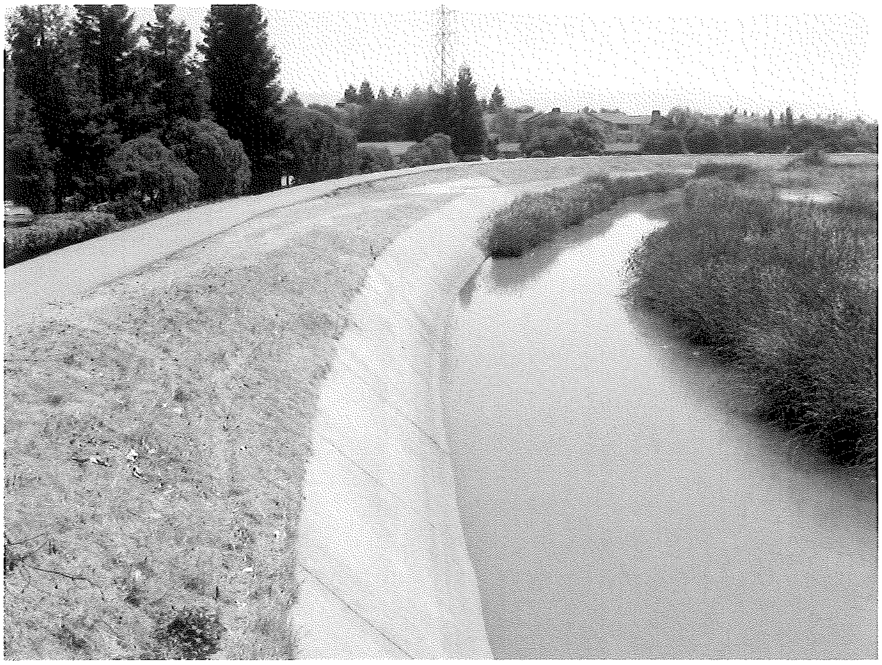
Photo ID	Location	Description
101, 102	39+20	N/S/~~B~~ views of inboard slope
103	40+10	N view of outboard slope from Milmont Ave.
104 -108	40+00	N view of channel and slopes from Milmont Avenue Bridge.
[illegible]	[illegible]	(compare E and West slopes to see sediment / blocked weep holes)
109-113	40+80	S view of channel and slopes from Bridge
114 - 119	0+90	View from opposite (west) bank (panorama)
120, 121, 122	41+30	Sediment Buildup under bridge (from W. bank)
123	40+80	So. view of outboard slope ([illegible] Swale area)
124 - 129	42+40	N/S views of inboard slope, crest, and outboard slope
130	41+90	N view, ravelling, bare inboard slope
131	43+00	Cracked Pavement and bare, ravelling slope.
132	0+00	Cracked / Damaged pavement
133 - 138	0+50	N/S views of inboard slope crest, and outboard slope
139 - 141, 142	2+30	N views of outboard, inboard slopes, channel
143	39+20	N view of outboard slope
144	39+00	S. view " " " "
145, 146	38+20	N. view of outboard slope (conc.) and retaining wall
~~146~~ 147	~~[illegible]~~ 37+60	S. view " " " (concrete).

SURVEY POINTS

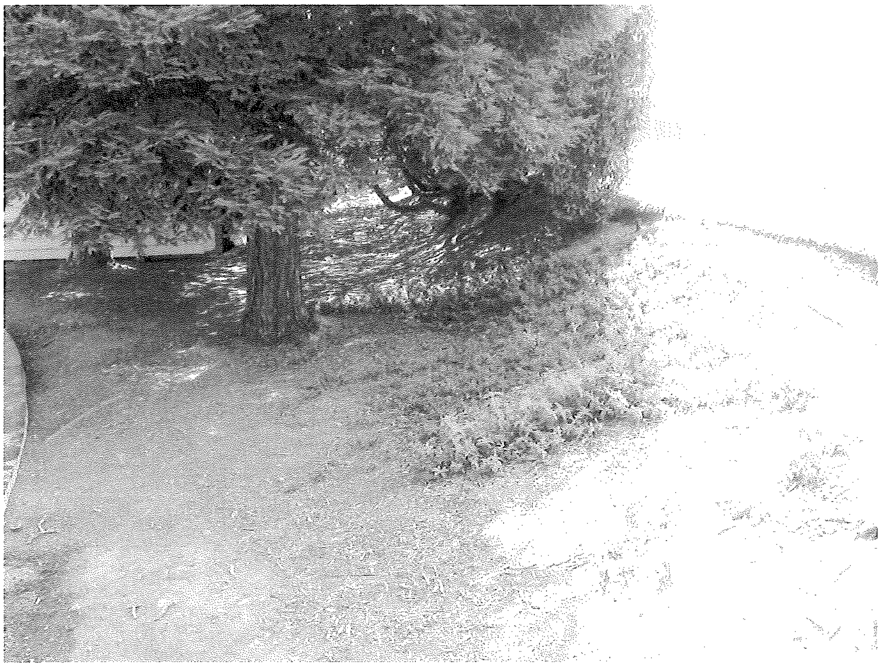
Point ID	Location	Description

ATTACHMENT 2

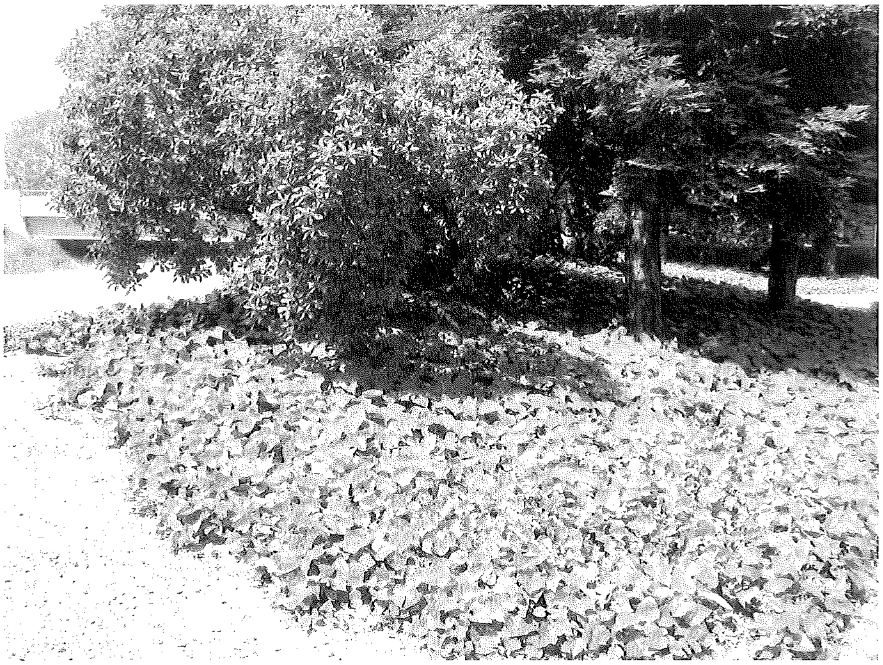
FIELD RECONNAISSANCE PHOTOS



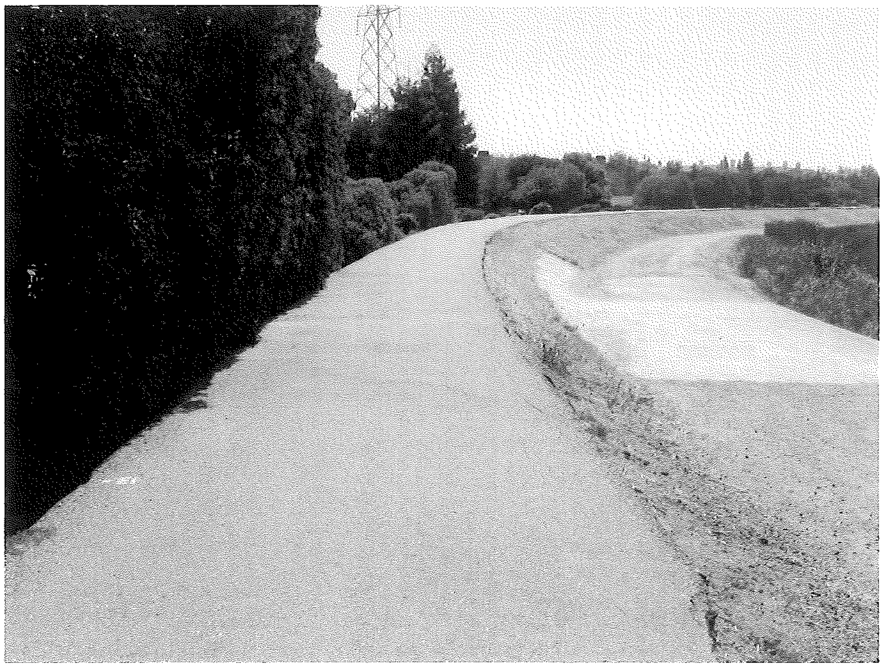
Photograph ID	Approximate Location (Project Station)	Description
001	11+40	East view from California Circle Bridge



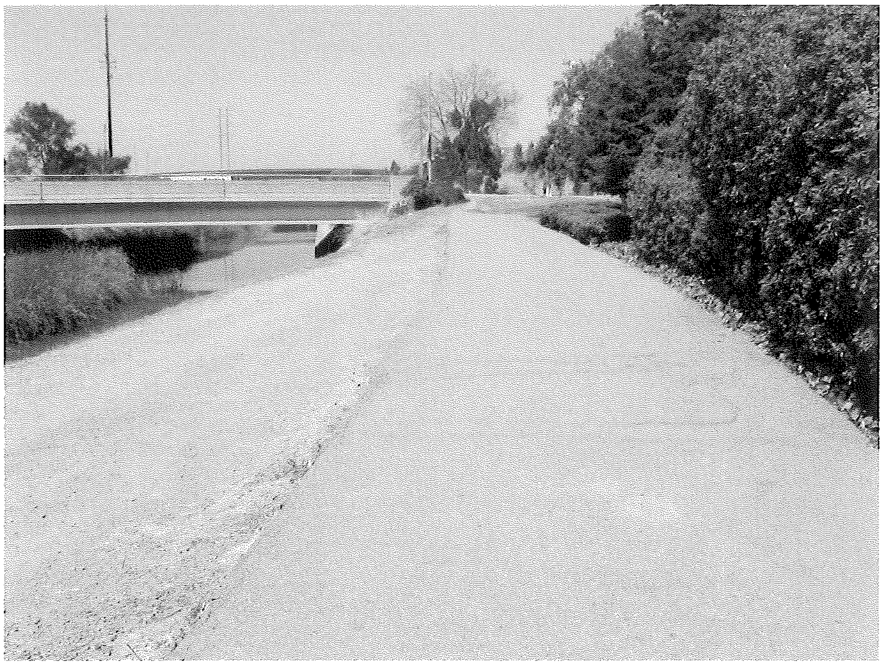
Photograph ID	Approximate Location (Project Station)	Description
002	12+80	East view of outboard levee slope



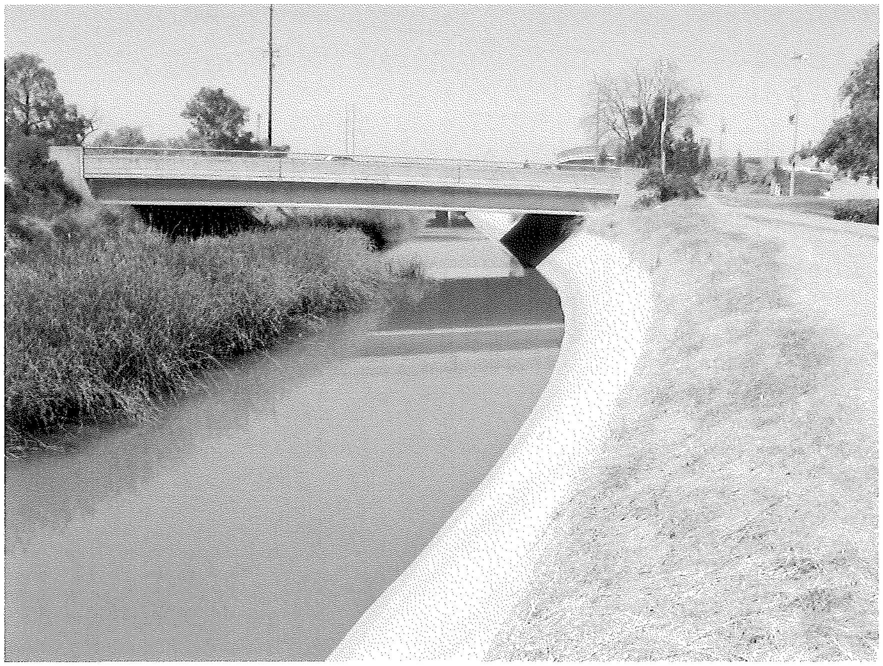
Photograph ID	Approximate Location (Project Station)	Description
003	12+80	West view of outboard levee slope



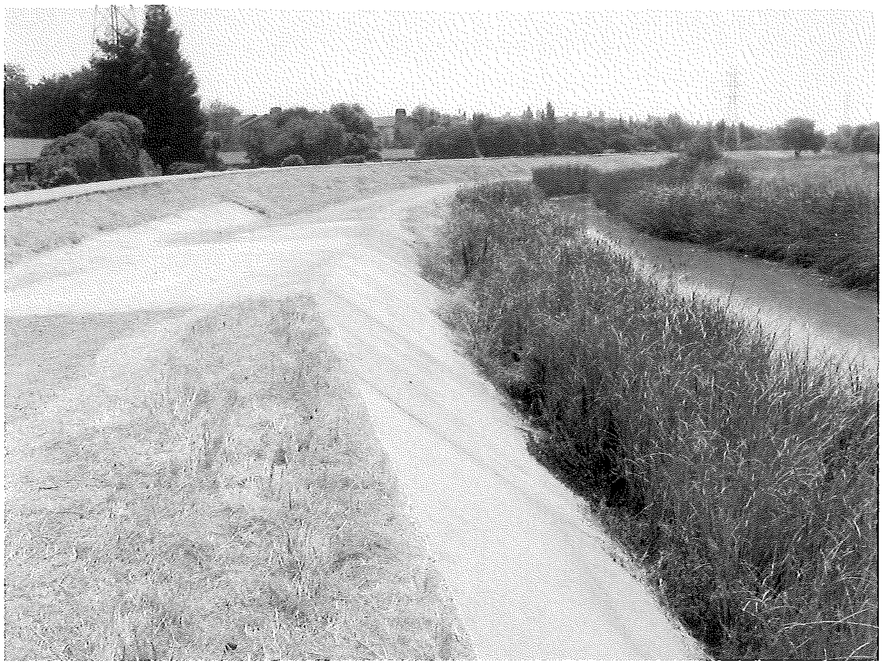
Photograph ID	Approximate Location (Project Station)	Description
004	12+80	East view of levee crest



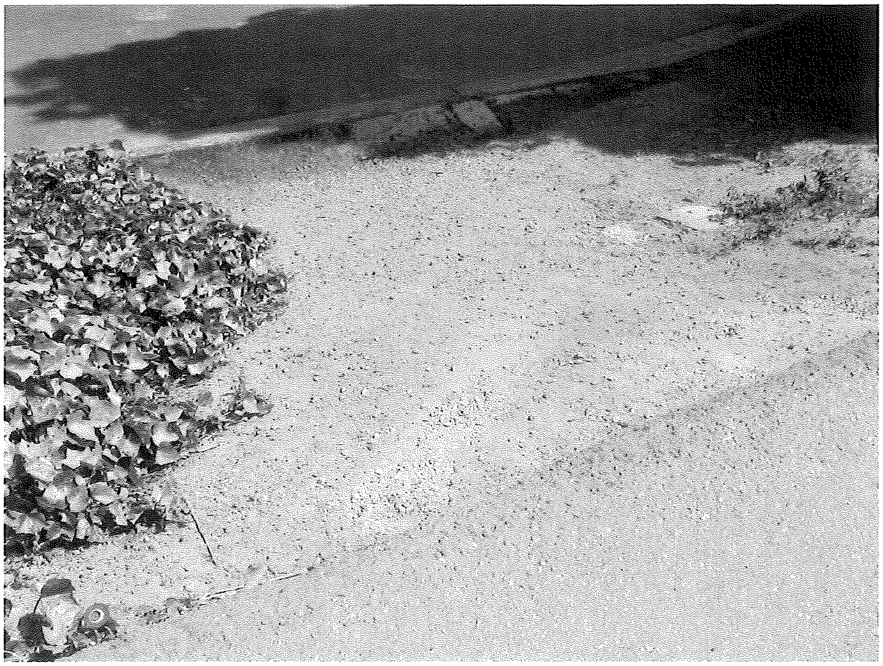
Photograph ID	Approximate Location (Project Station)	Description
005	12+80	West view of levee crest



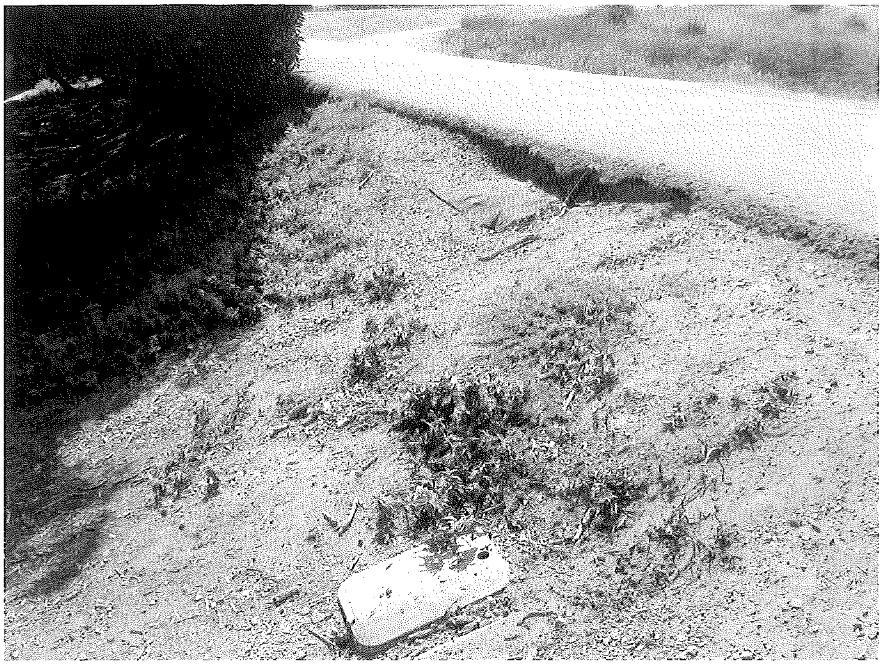
Photograph ID	Approximate Location (Project Station)	Description
006	12+80	West view of inboard levee slope



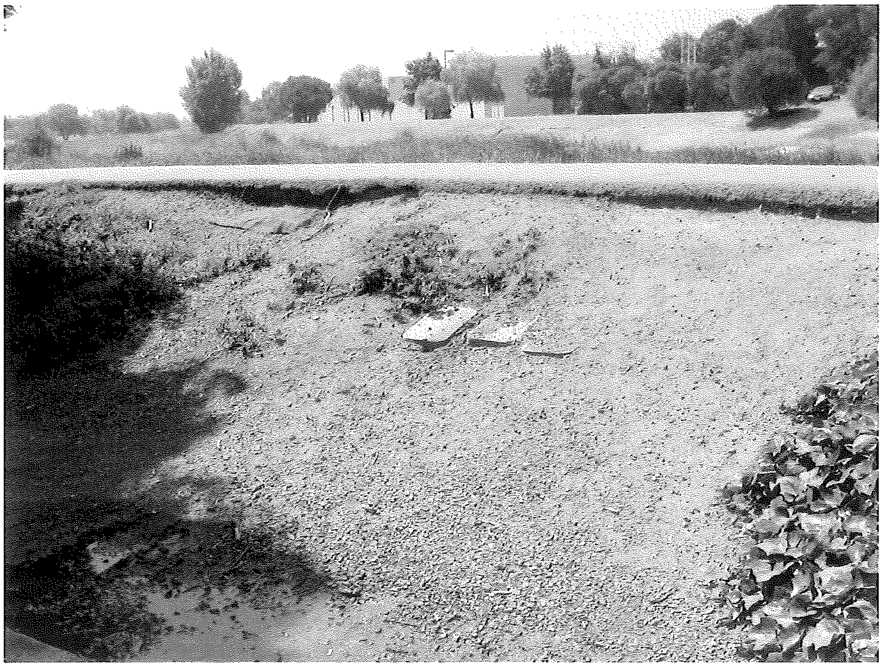
Photograph ID	Approximate Location (Project Station)	Description
007	12+80	East view of inboard levee slope



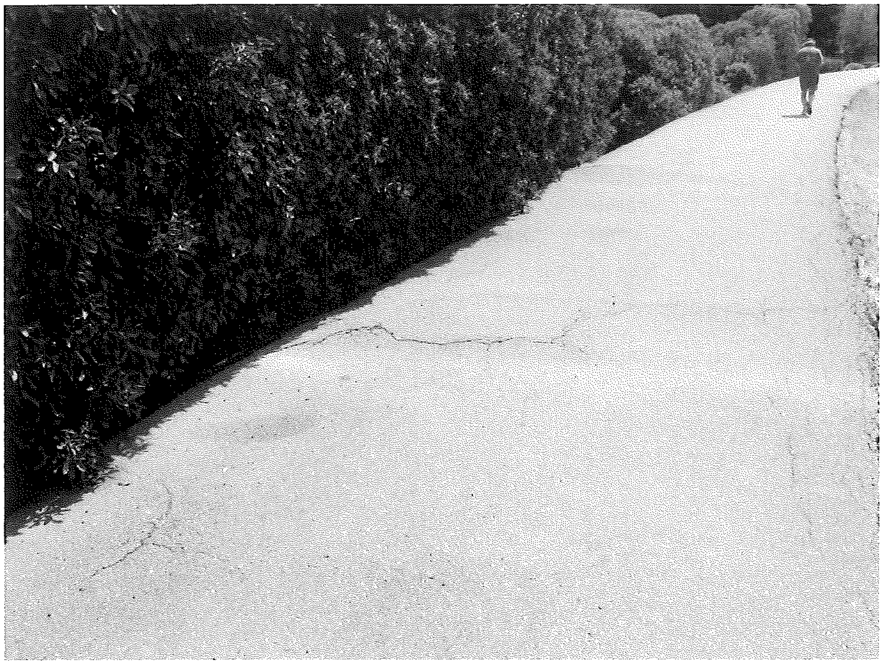
Photograph ID	Approximate Location (Project Station)	Description
008	12+80	Standing water at outboard levee toe, possible irrigation leak



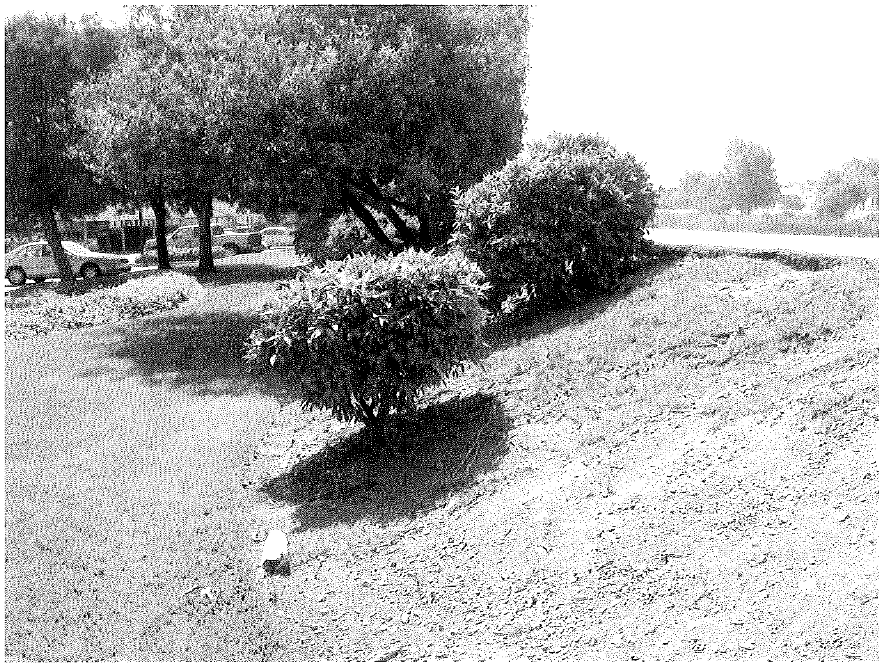
Photograph ID	Approximate Location (Project Station)	Description
009	12+80	Undermined pavement at levee access



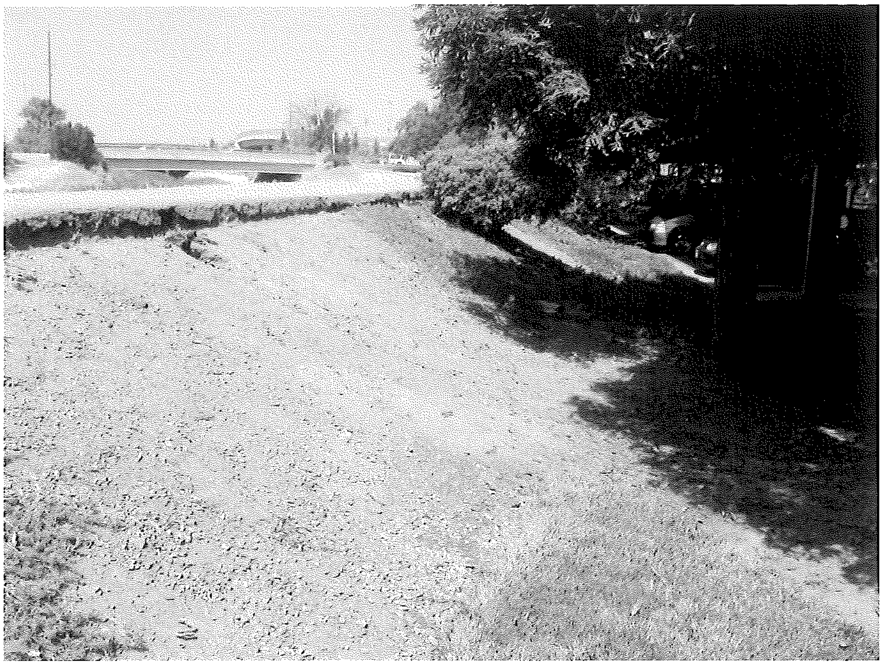
Photograph ID	Approximate Location (Project Station)	Description
010	12+80	Undermined pavement at levee access



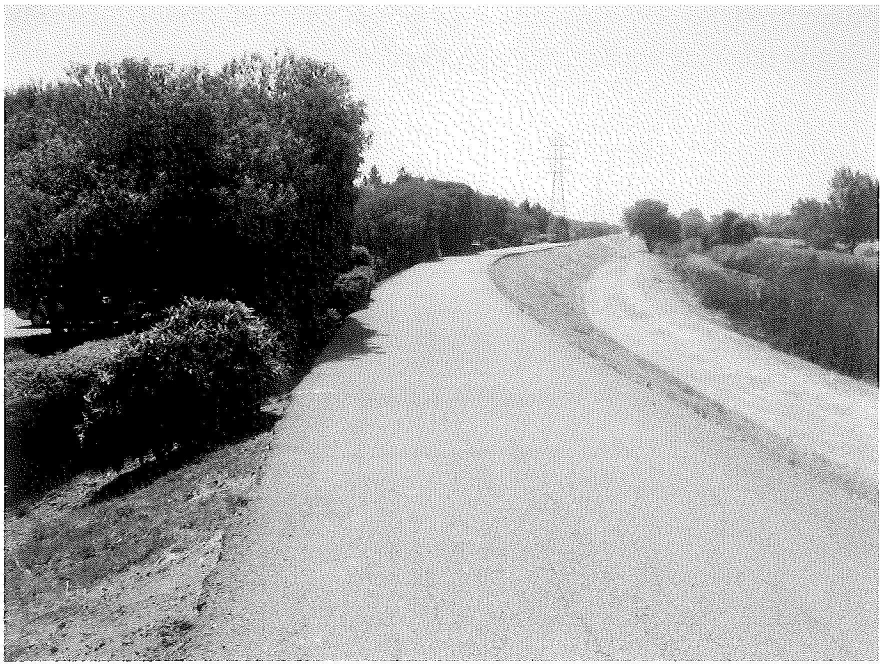
Photograph ID	Approximate Location (Project Station)	Description
011	13+10	Pavement cracks, possible from roots



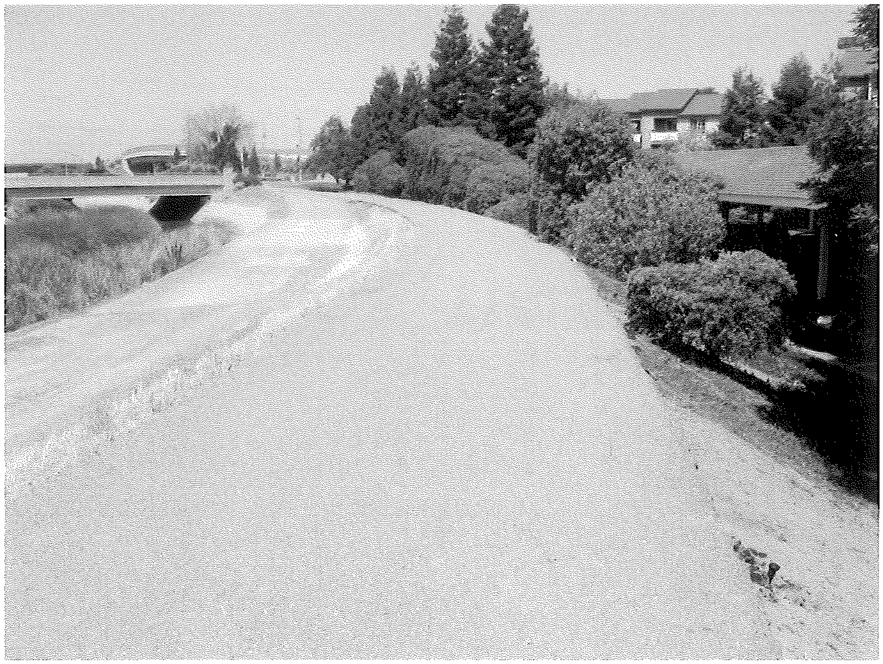
Photograph ID	Approximate Location (Project Station)	Description
012	14+80	East view of outboard levee slope



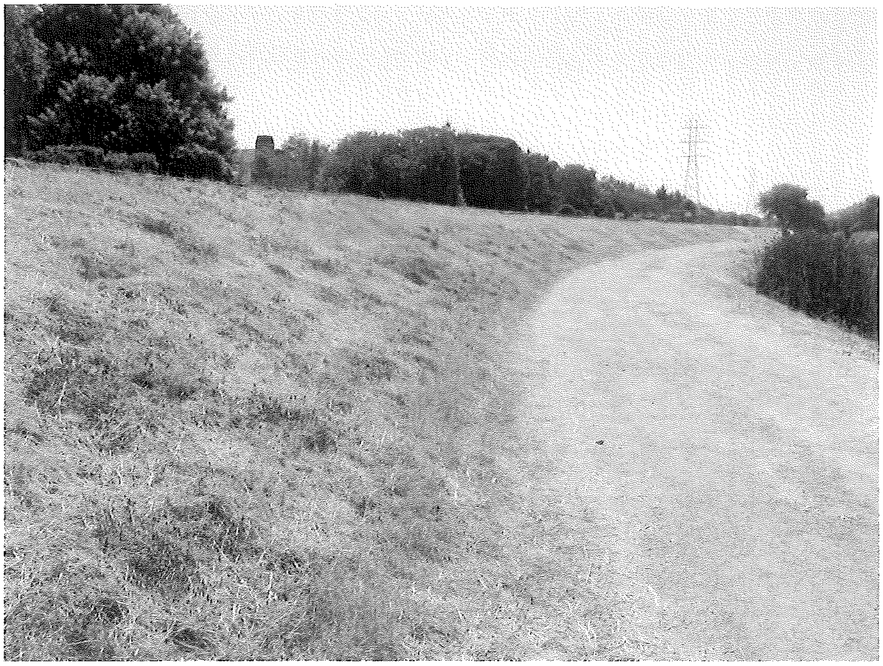
Photograph ID	Approximate Location (Project Station)	Description
013	14+80	West view of outboard levee slope



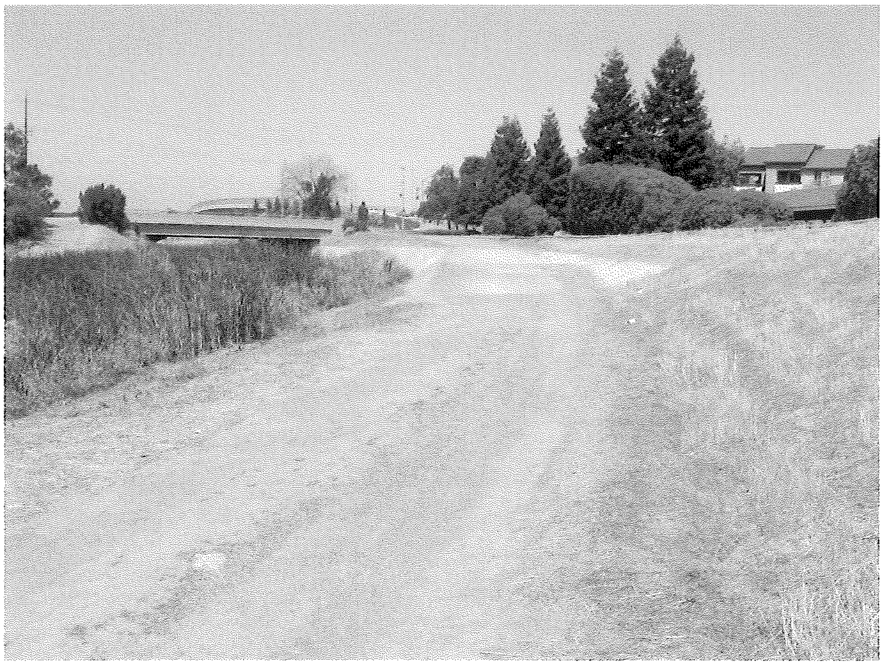
Photograph ID	Approximate Location (Project Station)	Description
014	14+80	East view of levee crest



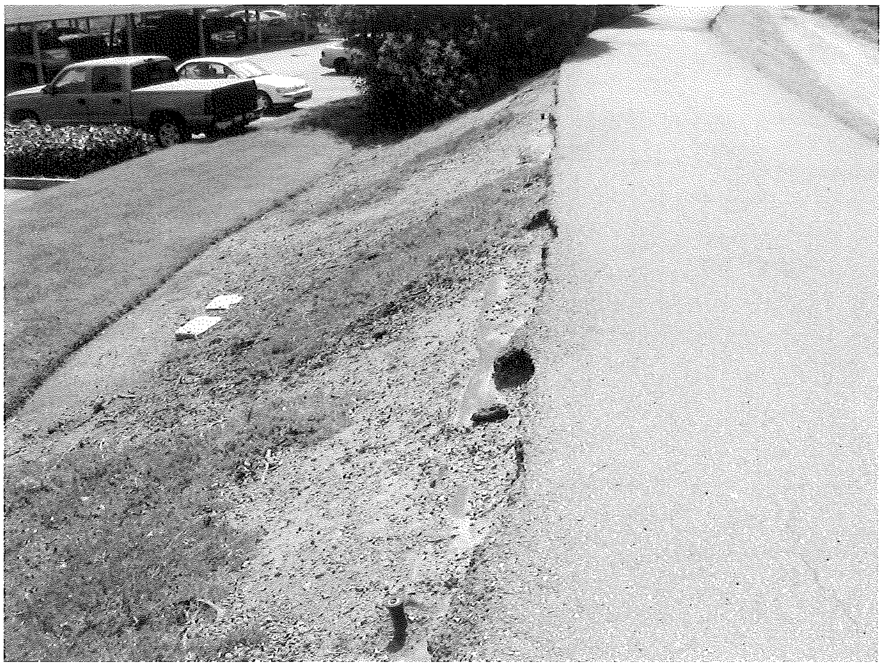
Photograph ID	Approximate Location (Project Station)	Description
015	14+80	West view of levee crest



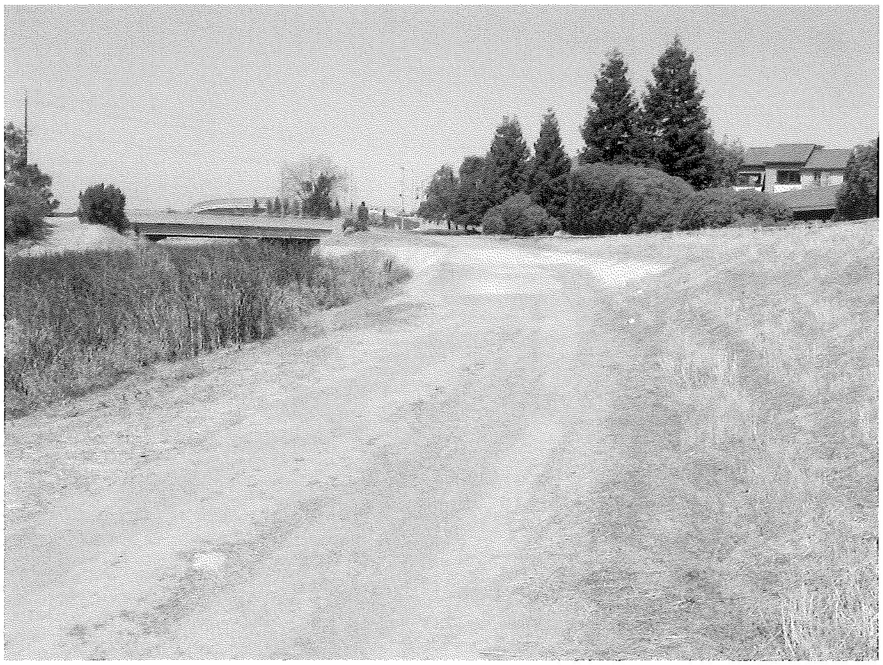
Photograph ID	Approximate Location (Project Station)	Description
016	14+80	East view of levee inboard slope



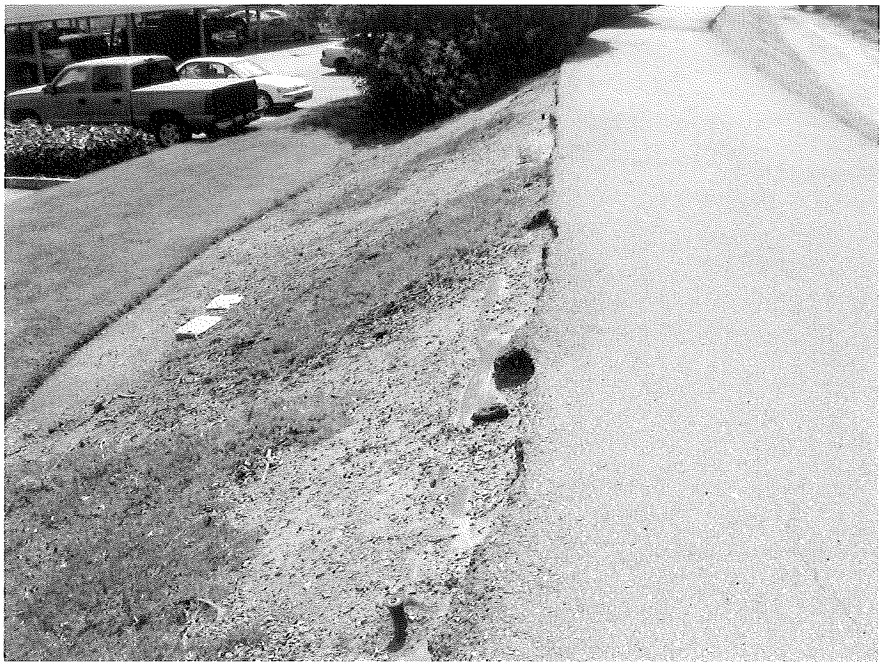
Photograph ID	Approximate Location (Project Station)	Description
017	14+80	West view of outboard levee slope



Photograph ID	Approximate Location (Project Station)	Description
018	15+60	Broken edge of pavement and irrigation system on outboard slope

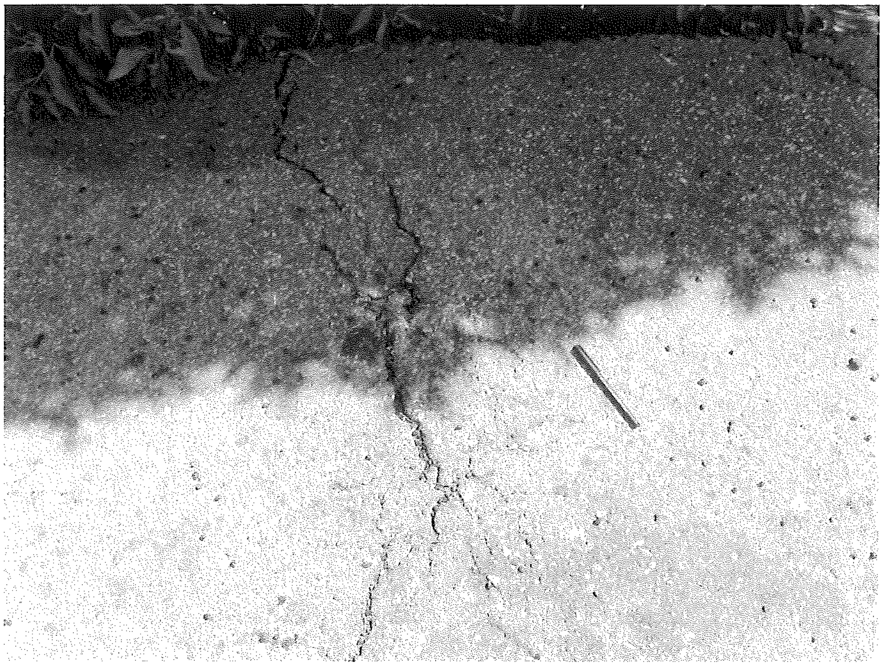


Photograph ID	Approximate Location (Project Station)	Description
017	14+80	West view of outboard levee slope

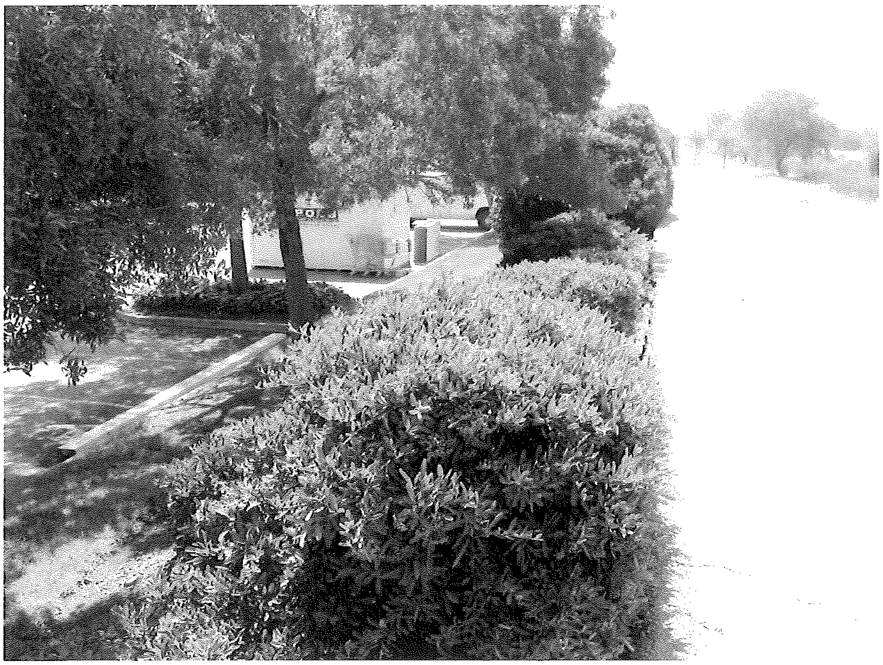


Photograph ID	Approximate Location (Project Station)	Description
018	15+60	Broken edge of pavement and irrigation system on outboard slope

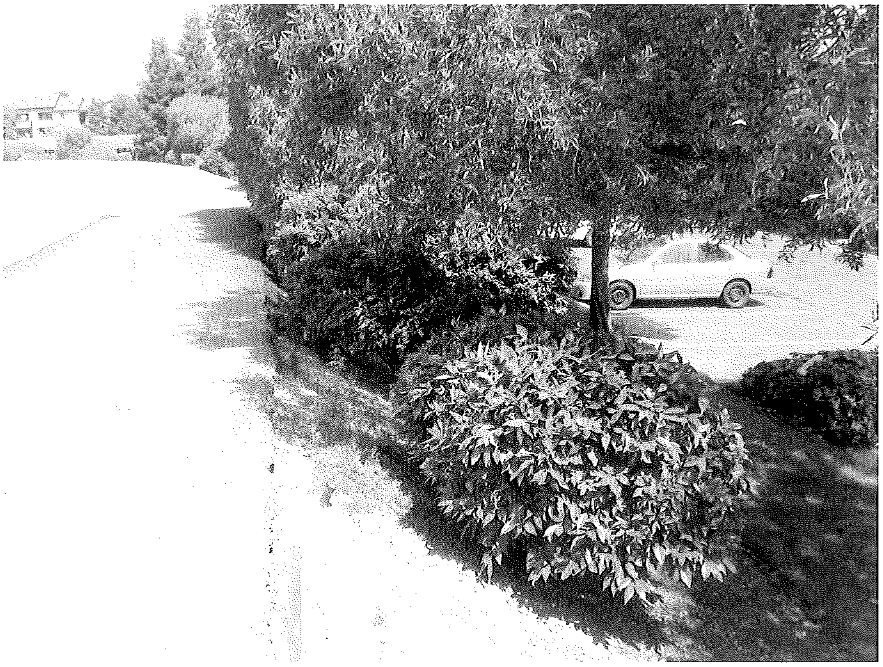
Photograph ID	Approximate Location (Project Station)	Description
019	16+10	Pavement cracks from roots



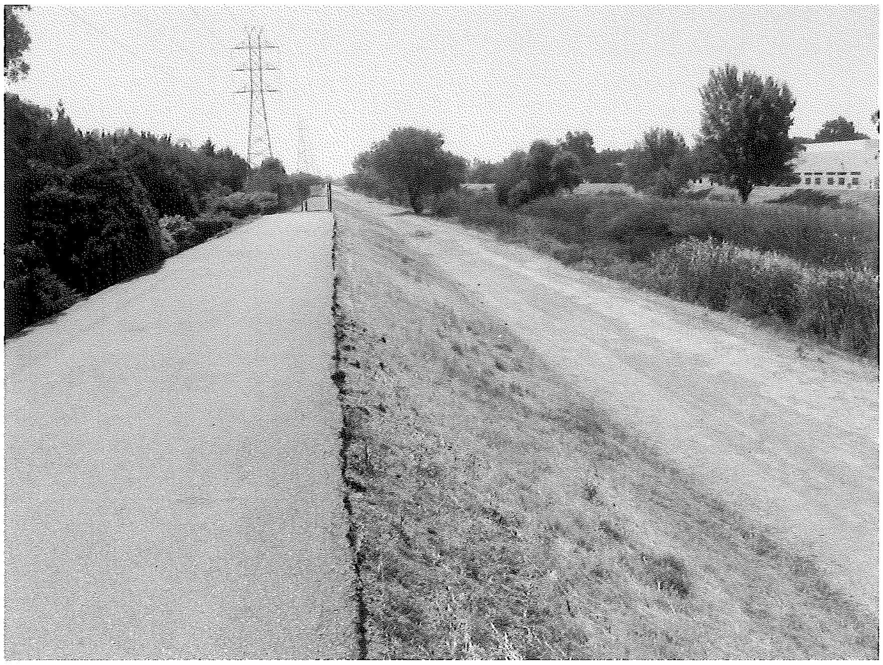
Photograph ID	Approximate Location (Project Station)	Description
020	16+10	Pavement cracks from roots



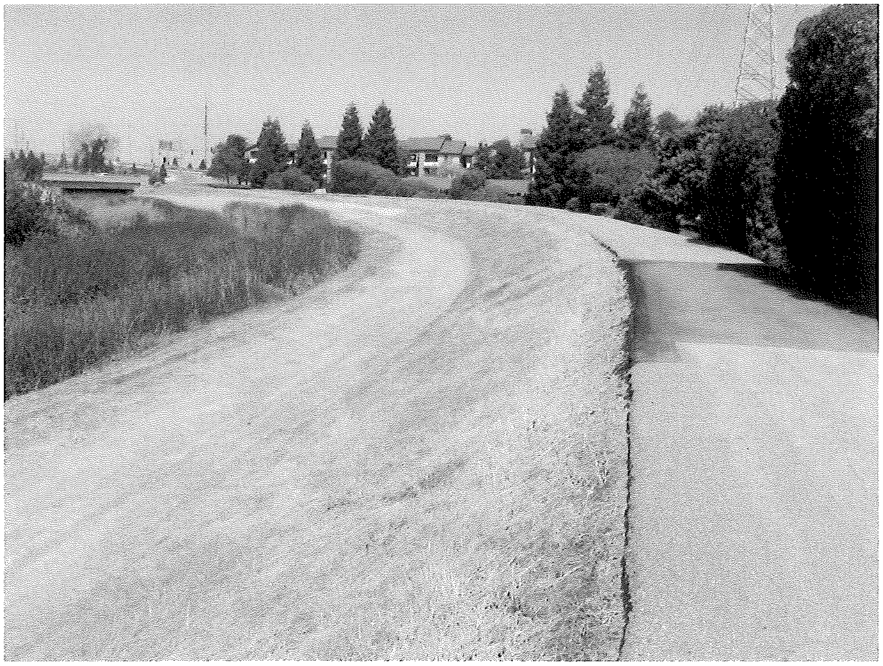
Photograph ID	Approximate Location (Project Station)	Description
021	17+00	South view of outboard levee slope



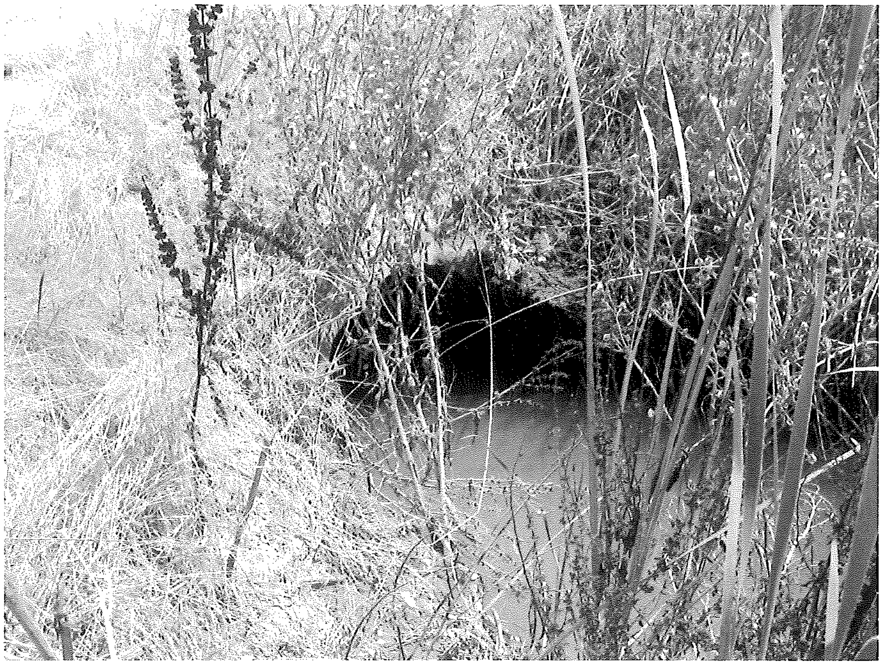
Photograph ID	Approximate Location (Project Station)	Description
022	17+00	North view of outboard levee slope



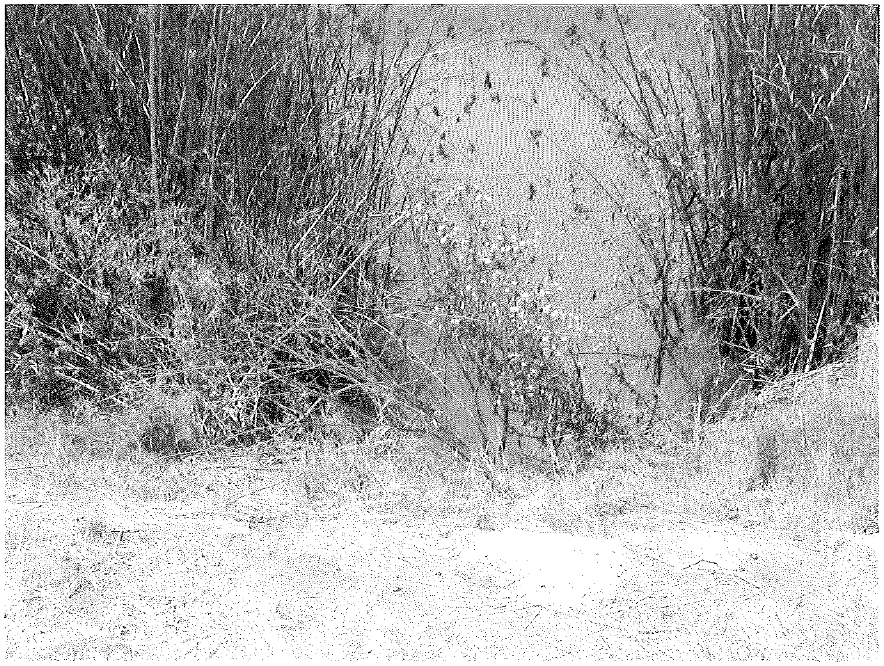
Photograph ID	Approximate Location (Project Station)	Description
023	17+00	South view of inboard levee slope



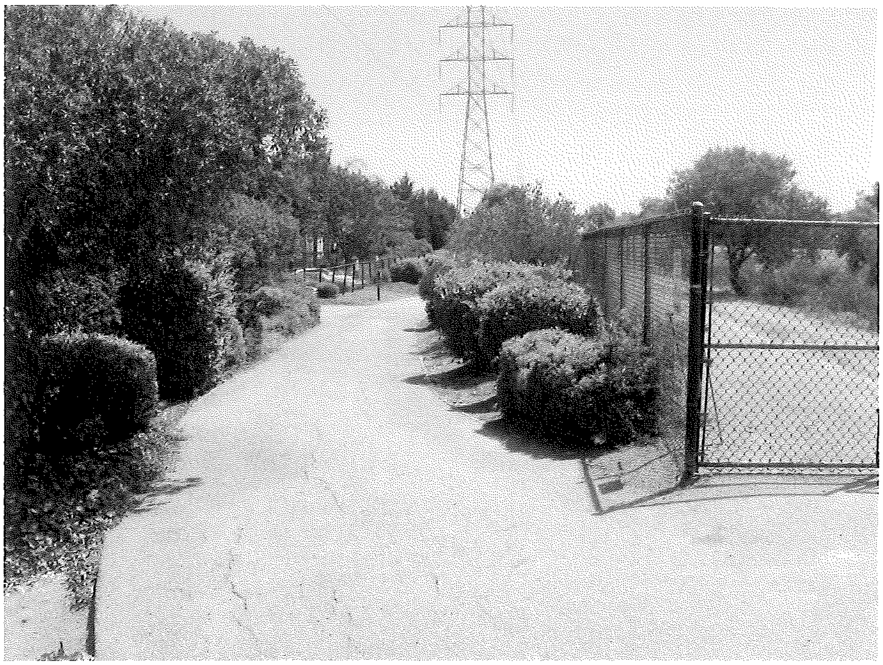
Photograph ID	Approximate Location (Project Station)	Description
024	17+00	North view of inboard levee slope



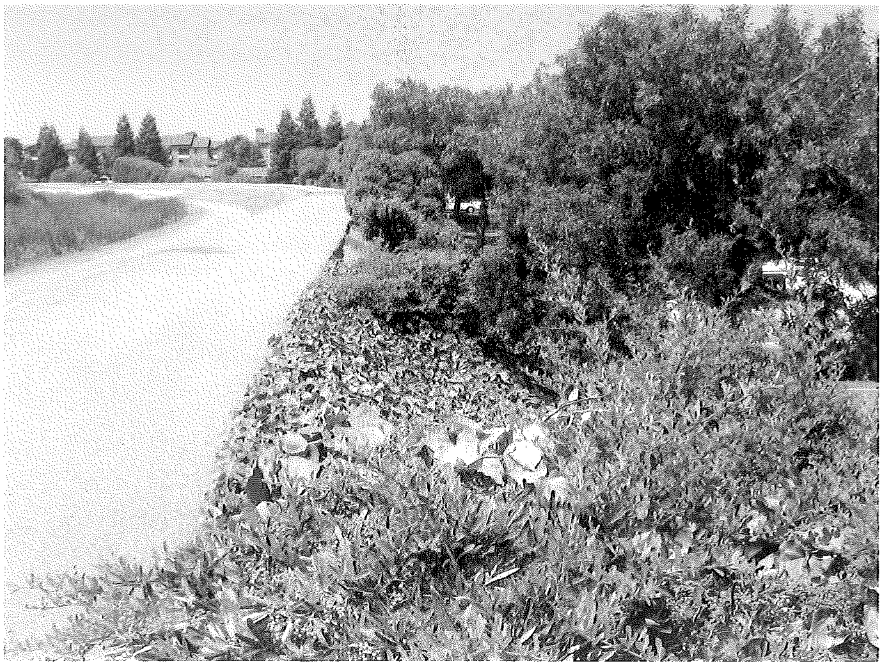
Photograph ID	Approximate Location (Project Station)	Description
025	26+00	Storm drain outlet (72" CMP about two-thirds submerged)



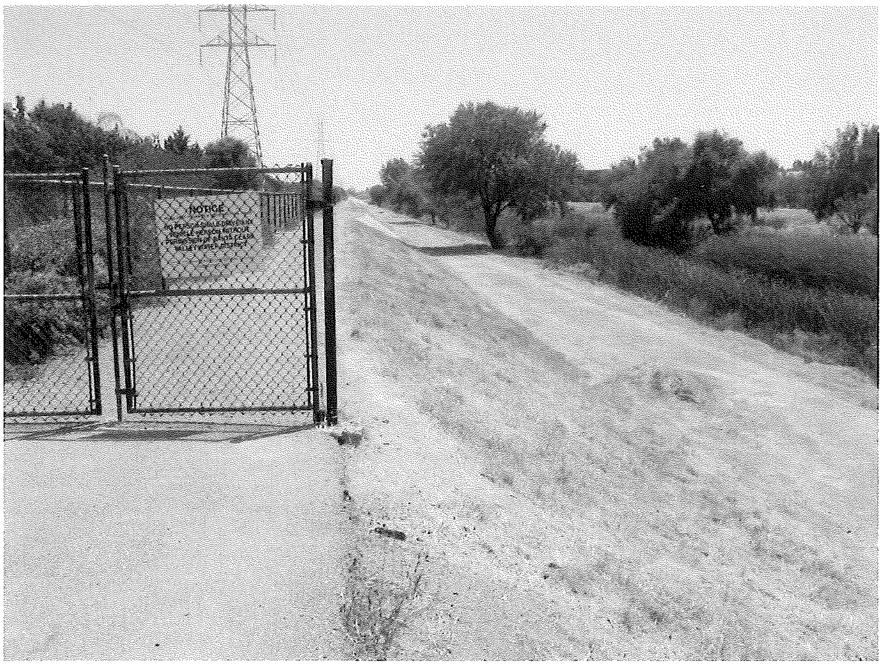
Photograph ID	Approximate Location (Project Station)	Description
026	26+00	Storm drain outlet (72" CMP about two-thirds submerged)



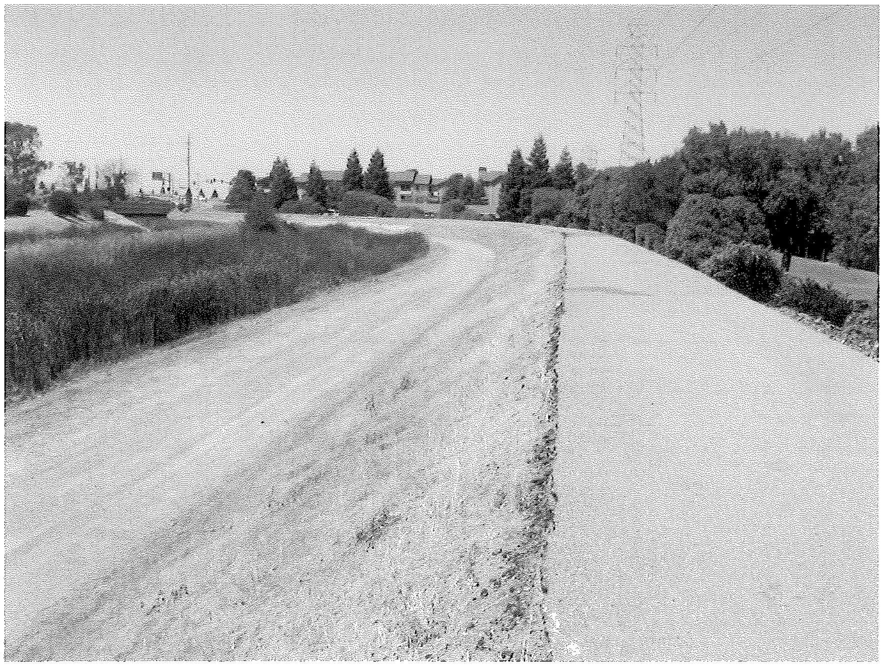
Photograph ID	Approximate Location (Project Station)	Description
027	18+80	South view of outboard levee slope



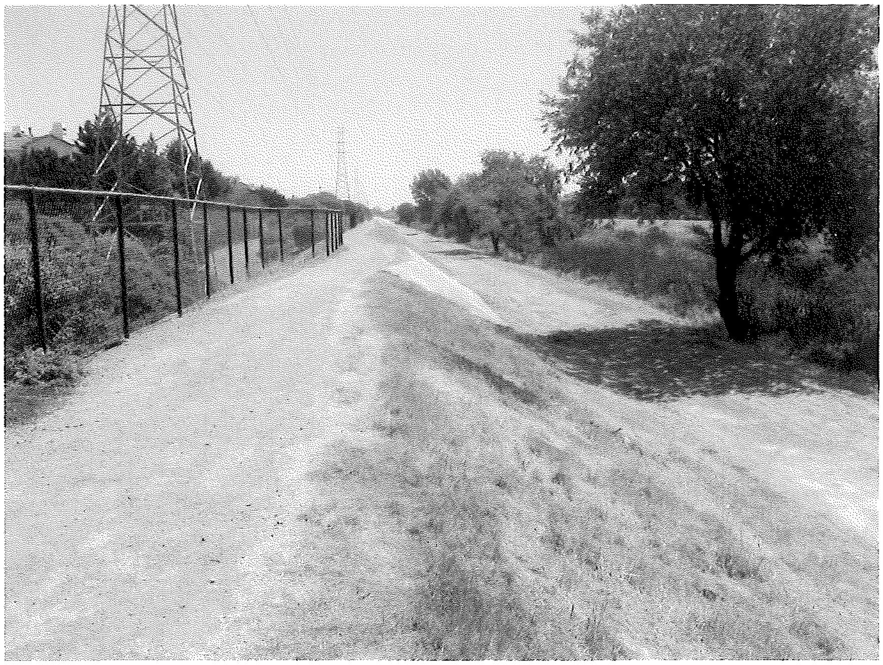
Photograph ID	Approximate Location (Project Station)	Description
028	18+80	North view of outboard levee slope



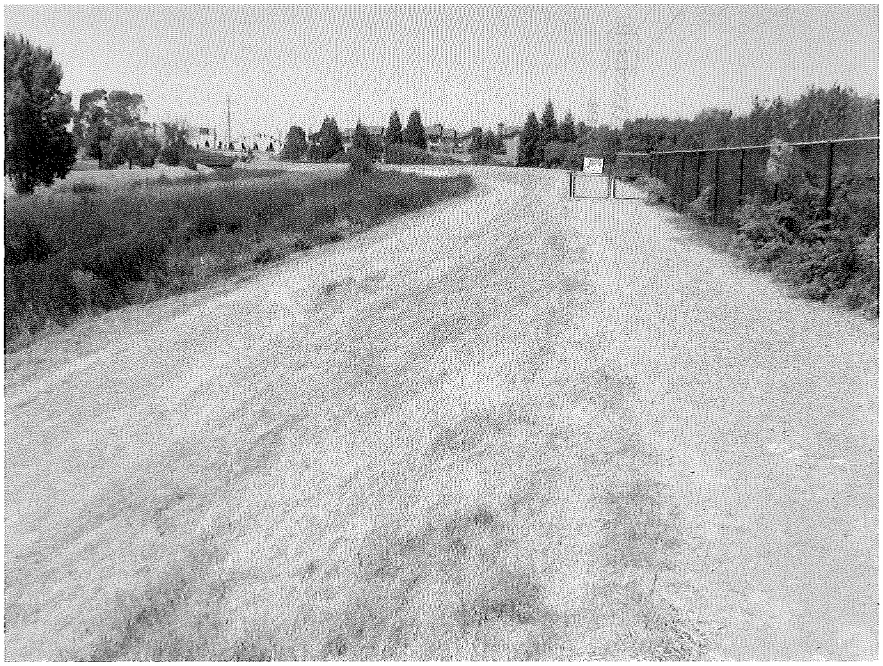
Photograph ID	Approximate Location (Project Station)	Description
029	18+80	South view of inboard levee slope



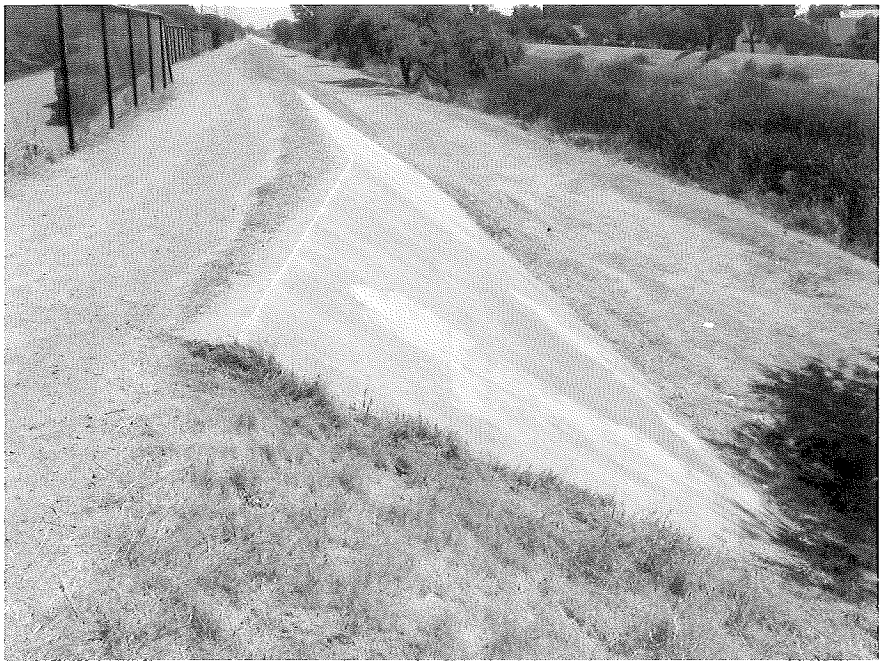
Photograph ID	Approximate Location (Project Station)	Description
030	18+80	North view of inboard levee slope



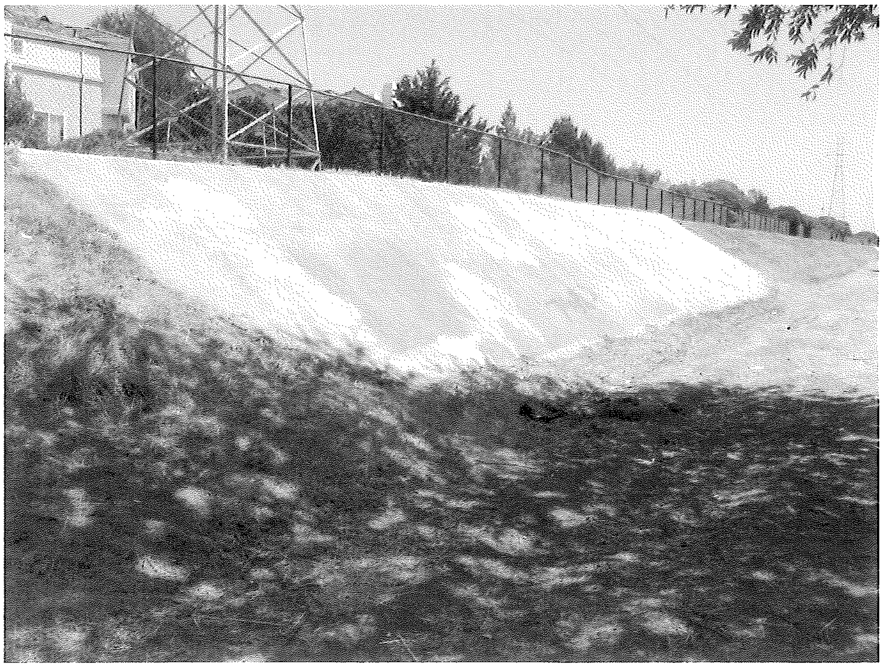
Photograph ID	Approximate Location (Project Station)	Description
031	20+00	South view of inboard levee slope



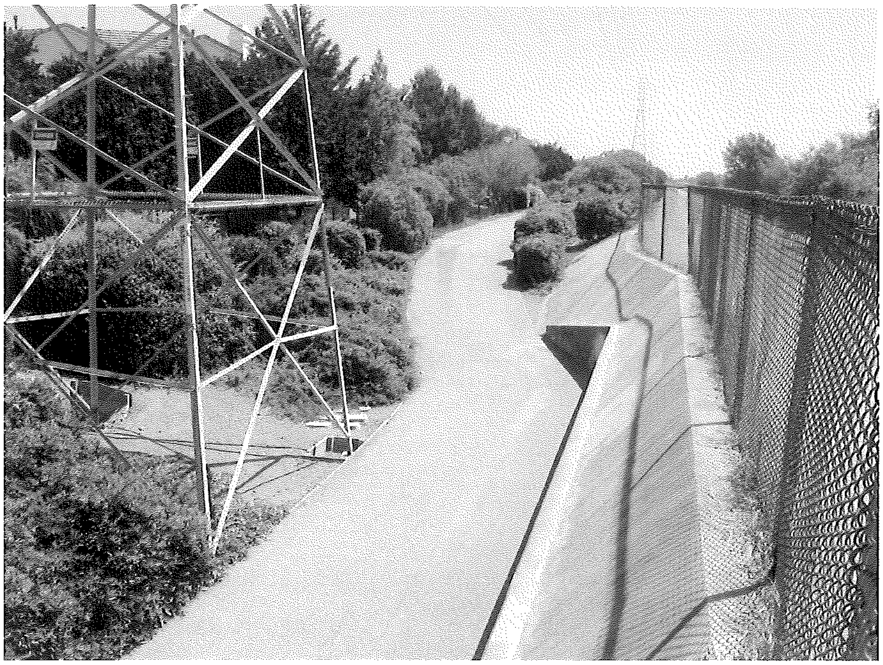
Photograph ID	Approximate Location (Project Station)	Description
032	20+00	North view of inboard levee slope



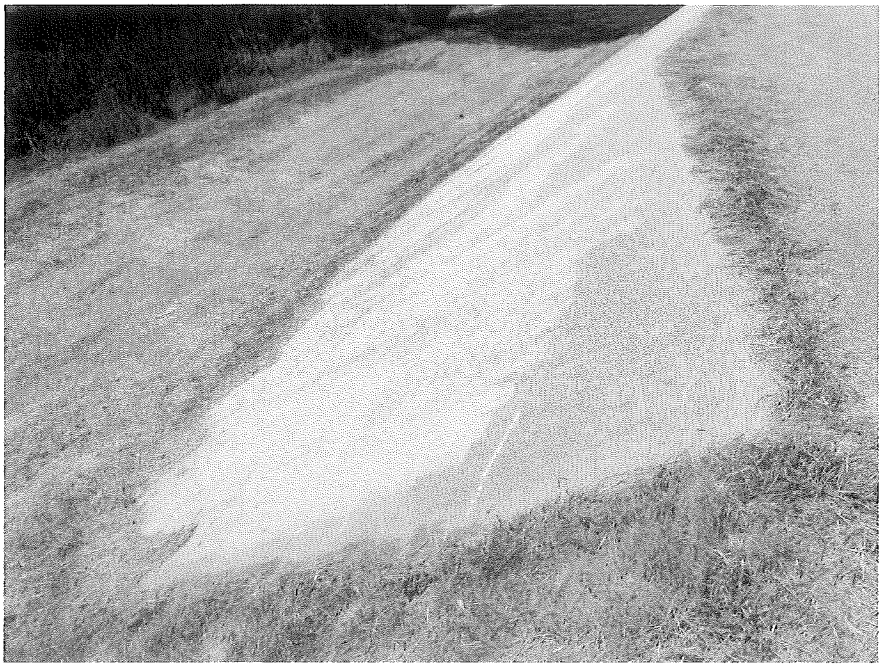
Photograph ID	Approximate Location (Project Station)	Description
033	20+80	South view of inboard levee concrete slope from crest



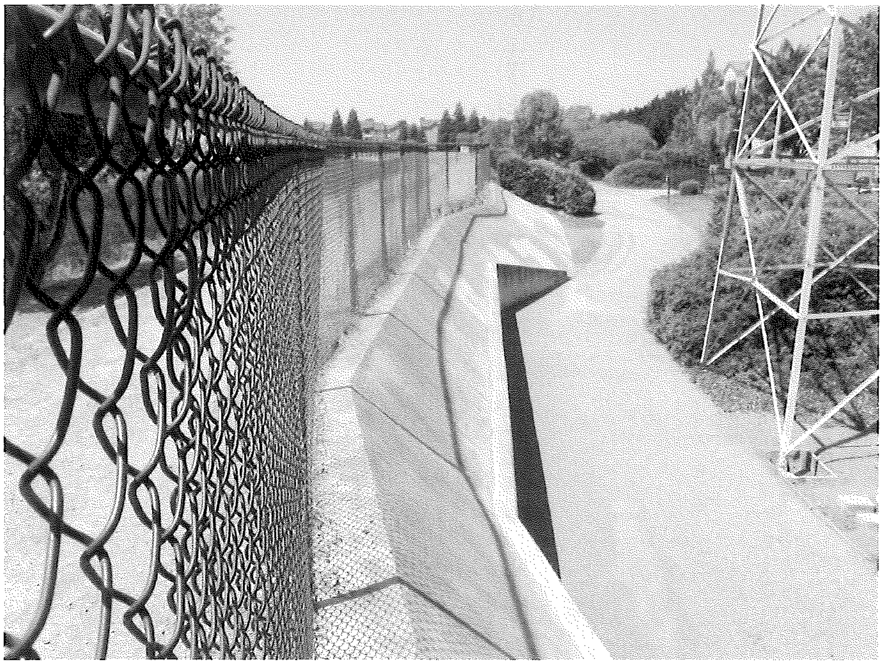
Photograph ID	Approximate Location (Project Station)	Description
034	20+80	South view of inboard concrete slope



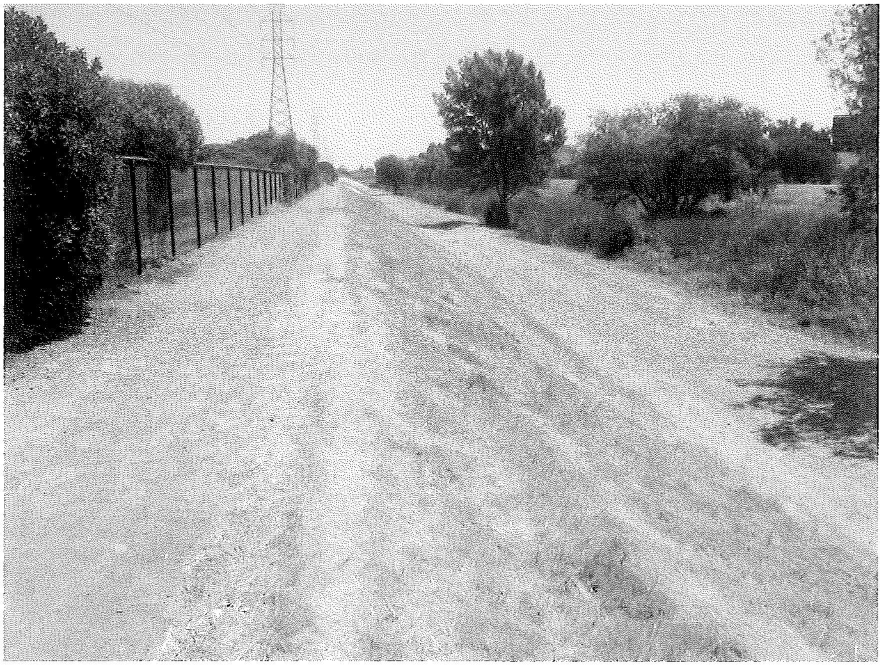
Photograph ID	Approximate Location (Project Station)	Description
035	21+00	South view of outboard retaining wall and electrical tower



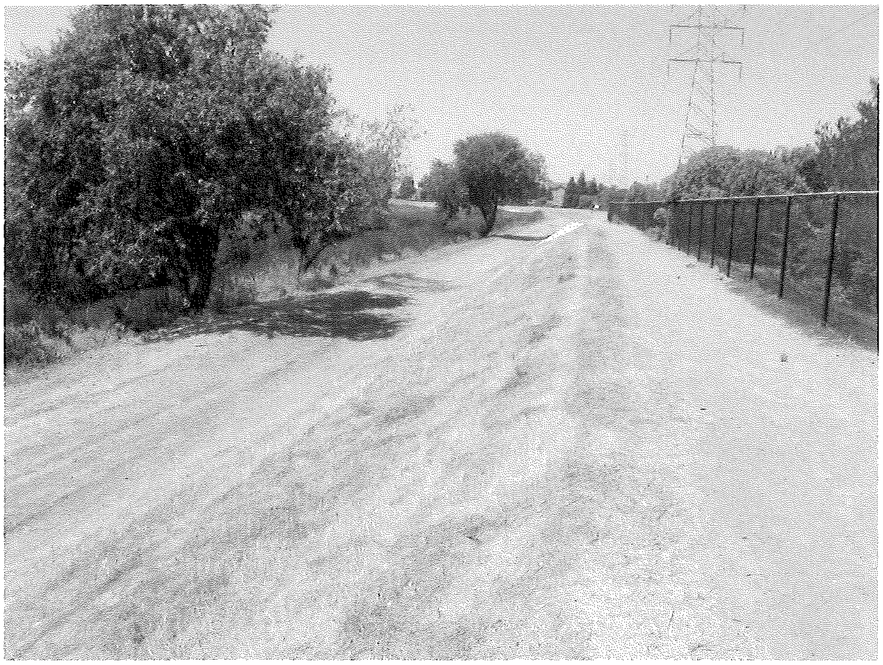
Photograph ID	Approximate Location (Project Station)	Description
036	21+80	North view of inboard concrete levee slope



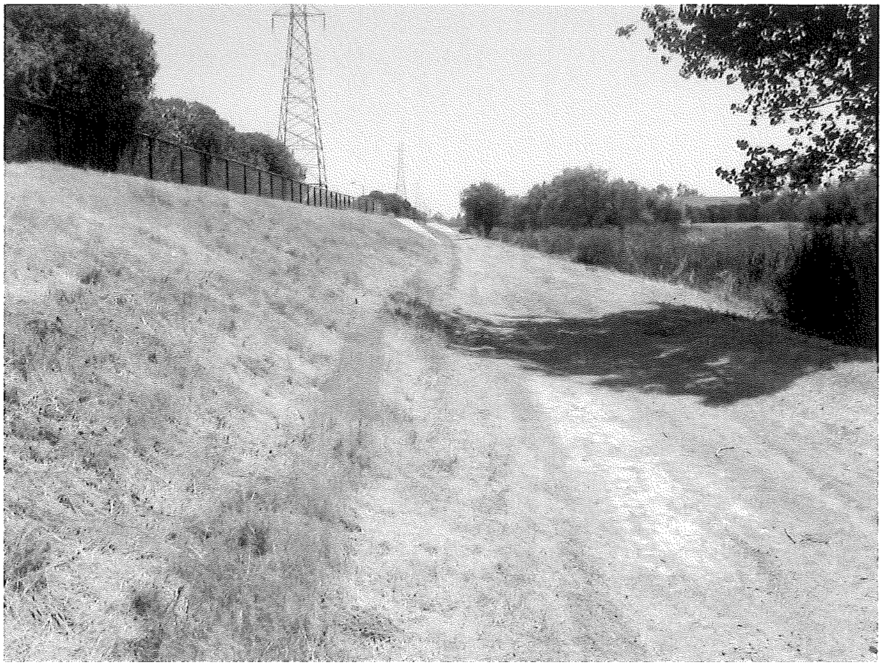
Photograph ID	Approximate Location (Project Station)	Description
037	21+80	North view of outboard concrete levee slope



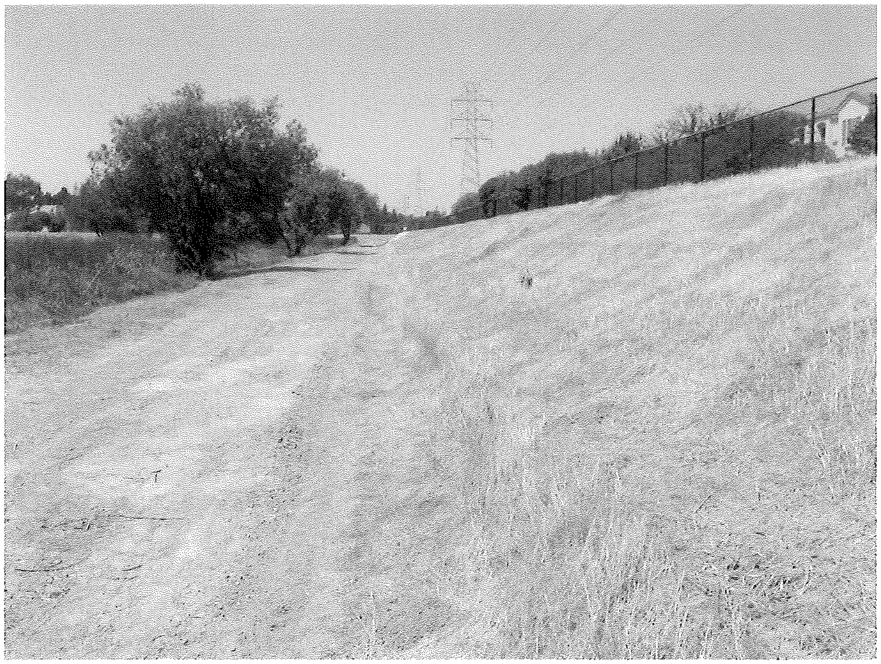
Photograph ID	Approximate Location (Project Station)	Description
038	24+00	South view of inboard levee slope and crest



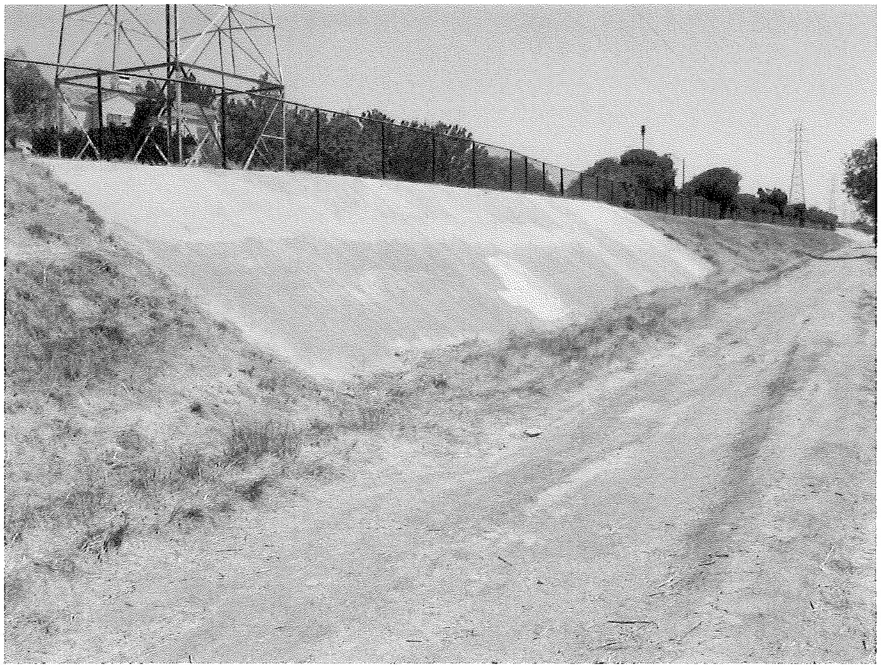
Photograph ID	Approximate Location (Project Station)	Description
039	24+00	North view of inboard levee slope and crest



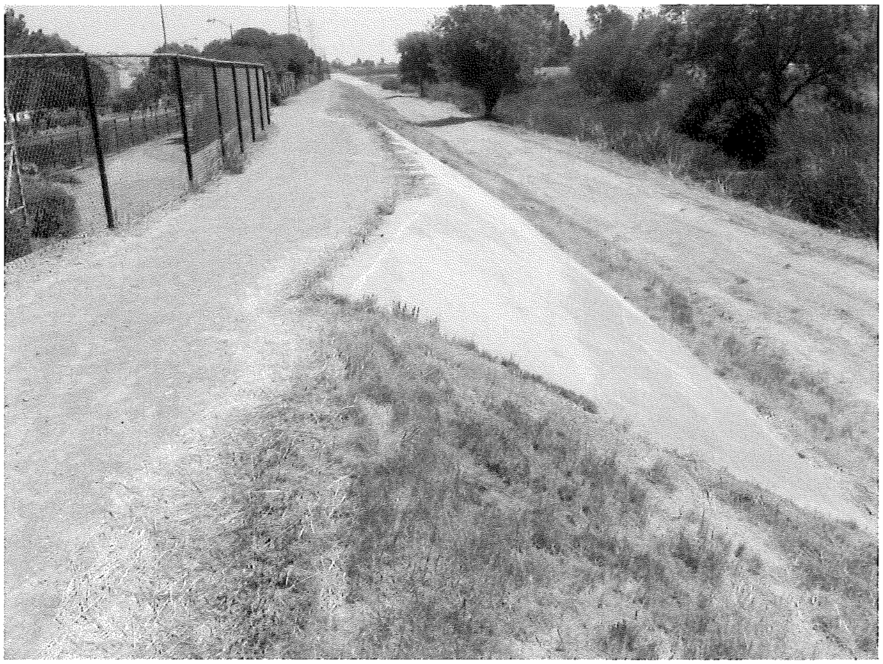
Photograph ID	Approximate Location (Project Station)	Description
040	26+00	South view of inboard levee slope from maintenance road



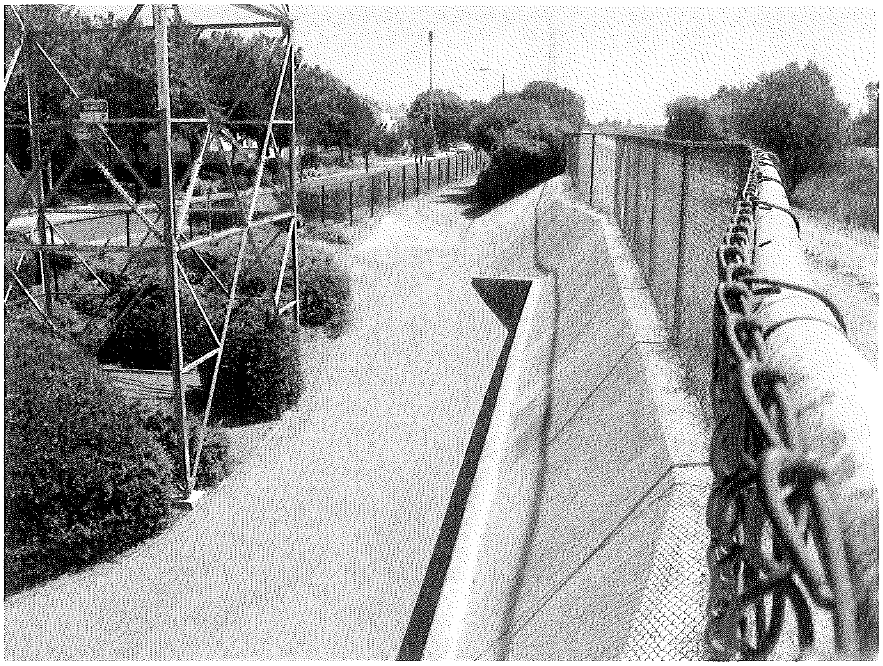
Photograph ID	Approximate Location (Project Station)	Description
041	26+00	North view of inboard levee slope from maintenance road



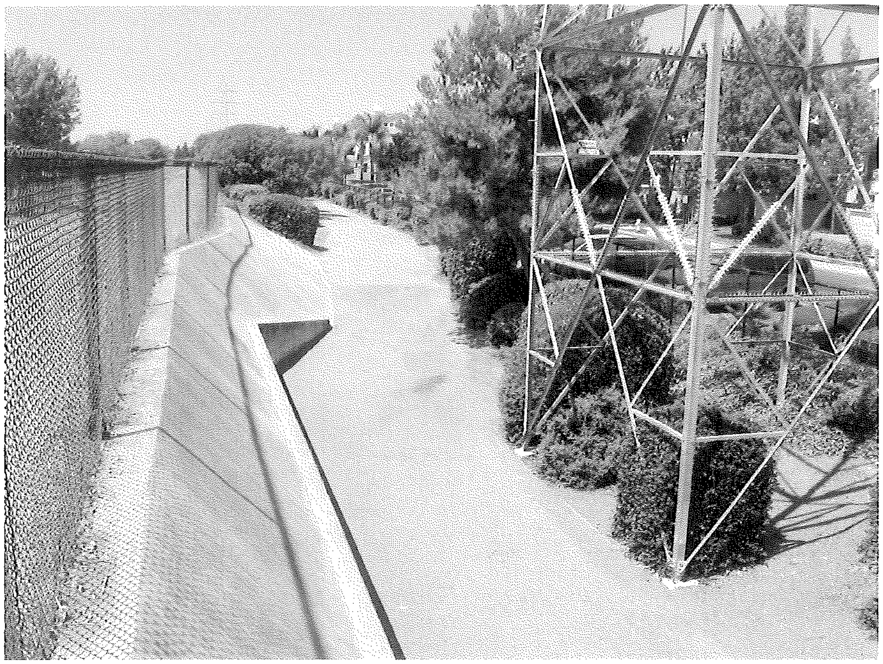
Photograph ID	Approximate Location (Project Station)	Description
042	28+50	South view of inboard concrete slope



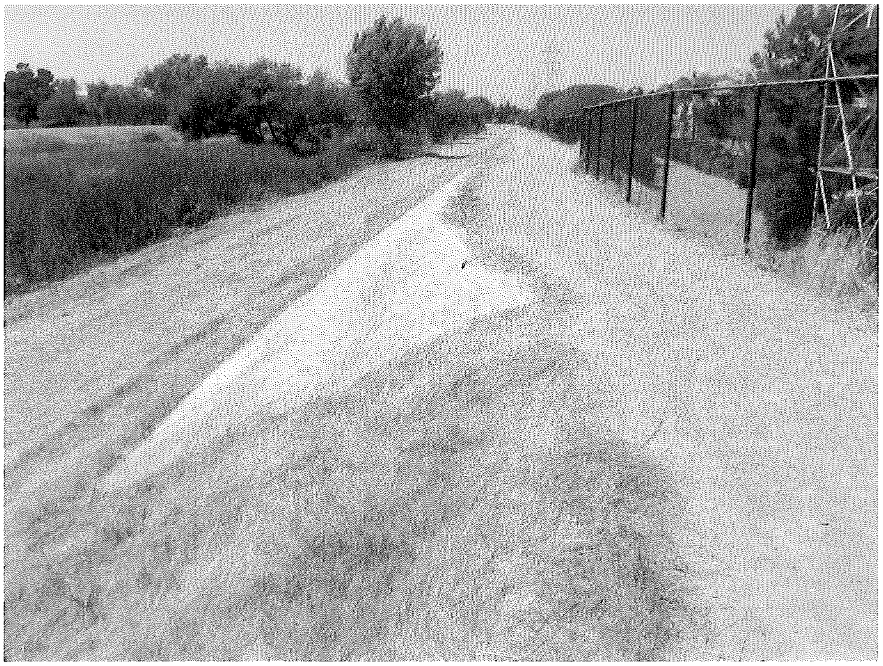
Photograph ID	Approximate Location (Project Station)	Description
043	28+50	South view of inboard levee concrete slope and crest



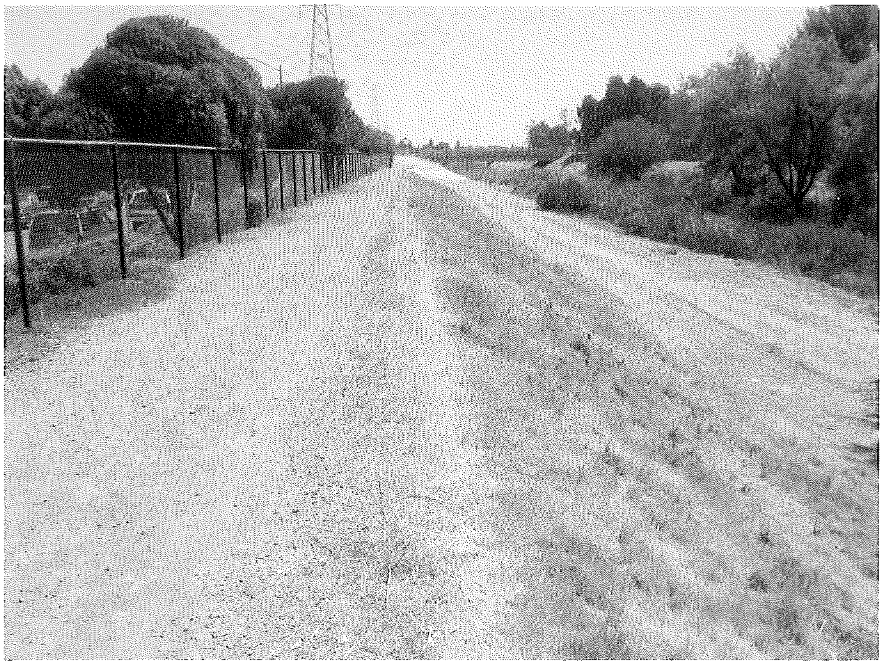
Photograph ID	Approximate Location (Project Station)	Description
044	28+70	South view of outboard levee retaining wall and electrical tower



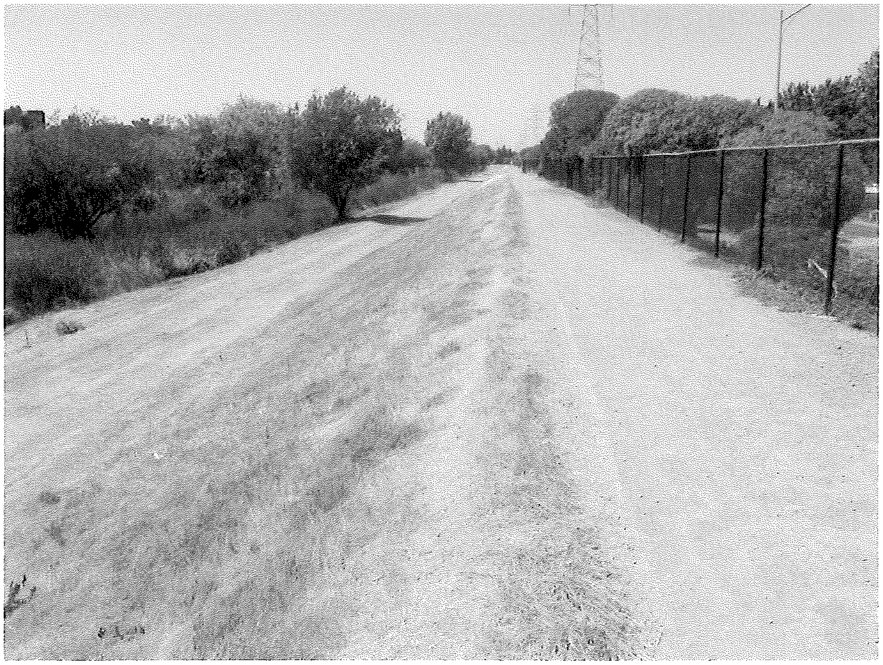
Photograph ID	Approximate Location (Project Station)	Description
045	29+60	North view of outboard levee retaining wall and electrical tower



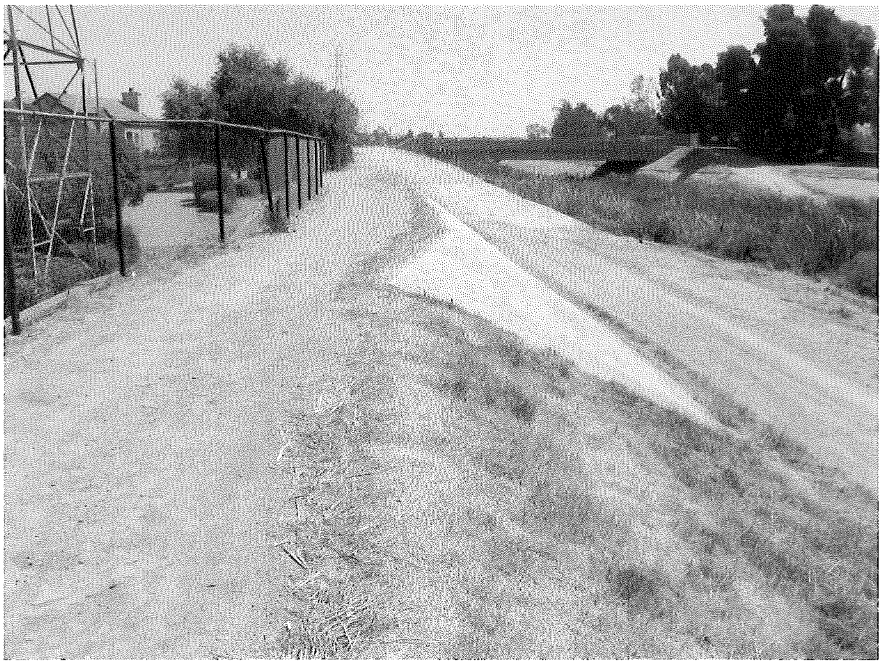
Photograph ID	Approximate Location (Project Station)	Description
046	29+90	North view of inboard concrete slope and crest



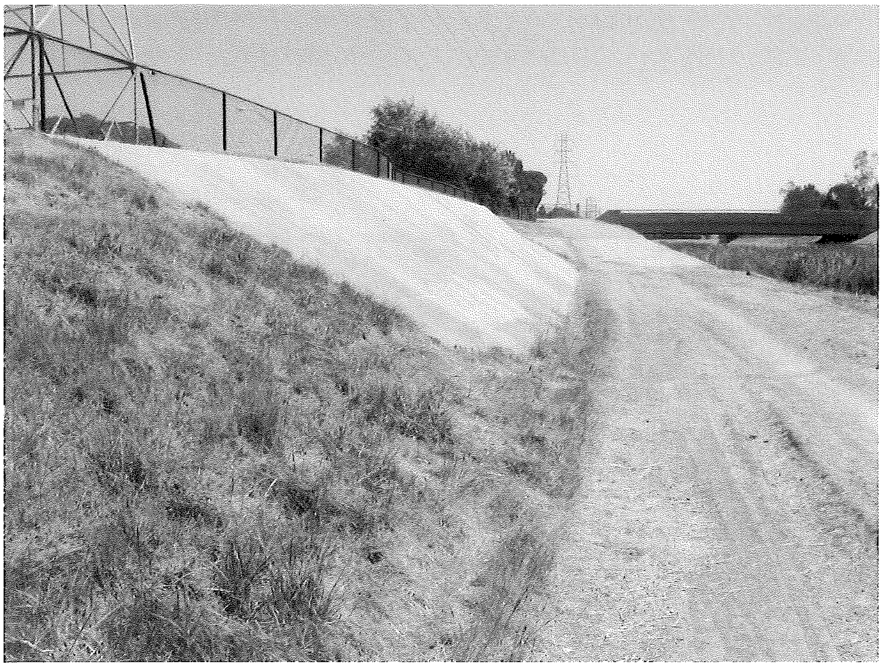
Photograph ID	Approximate Location (Project Station)	Description
047	33+00	South view of inboard levee slope and crest



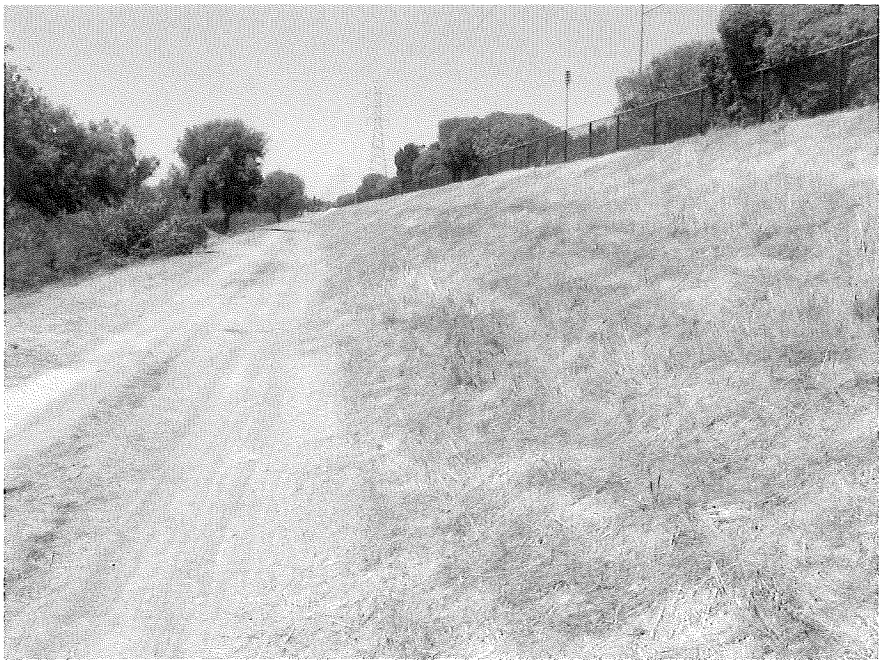
Photograph ID	Approximate Location (Project Station)	Description
048	33+00	North view of inboard levee slope and crest



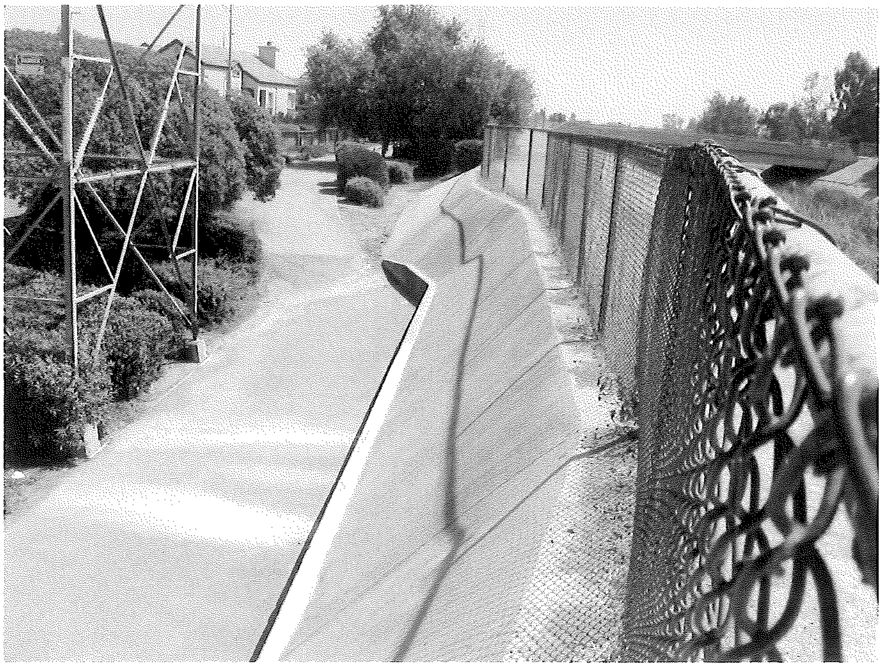
Photograph ID	Approximate Location (Project Station)	Description
049	36+20	South view of concrete slope and crest



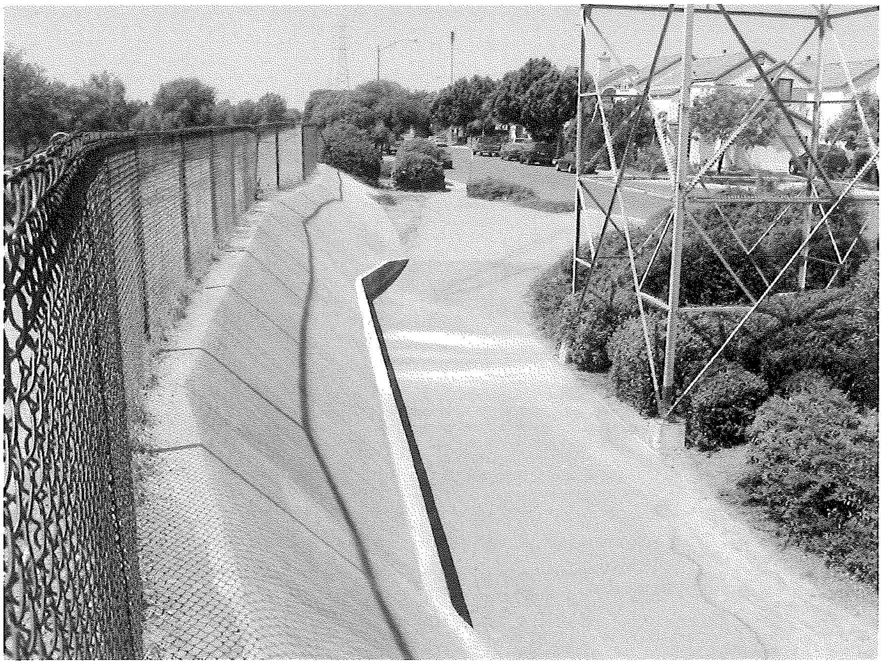
Photograph ID	Approximate Location (Project Station)	Description
050	36+20	South view of inboard levee slope from maintenance road



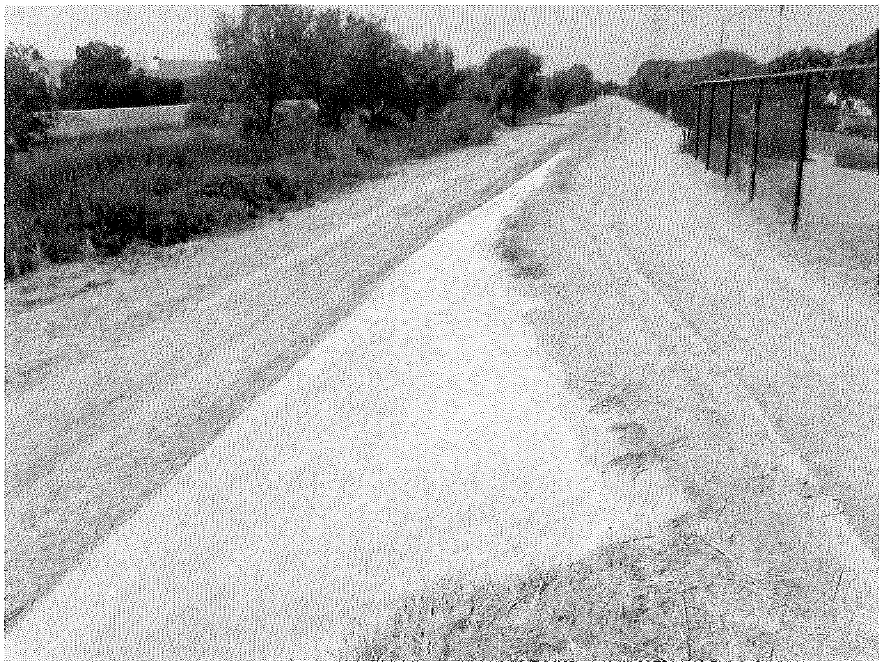
Photograph ID	Approximate Location (Project Station)	Description
051	36+20	North view of inboard levee slope from maintenance road



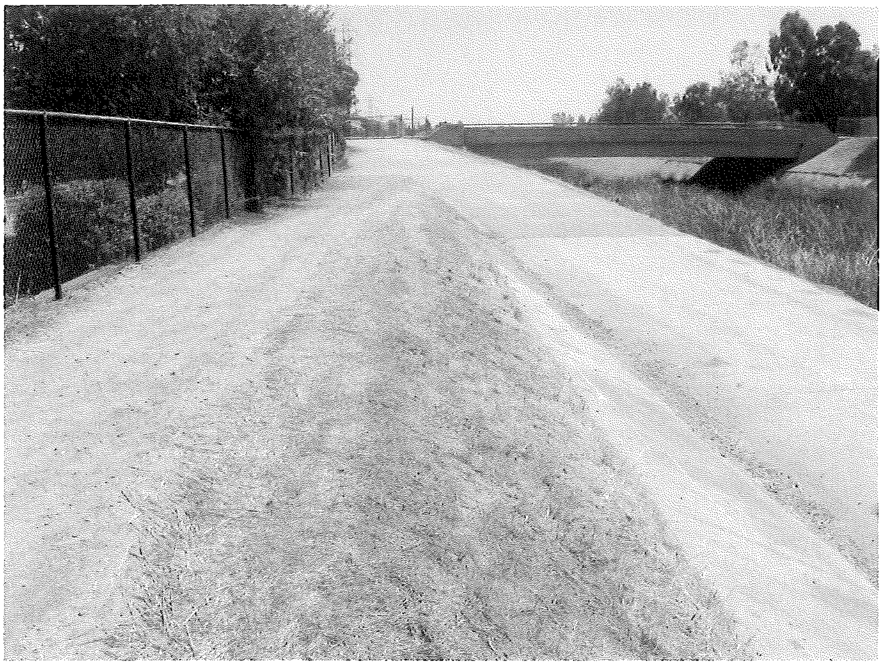
Photograph ID	Approximate Location (Project Station)	Description
052	36+60	South view of outboard retaining wall and electrical tower



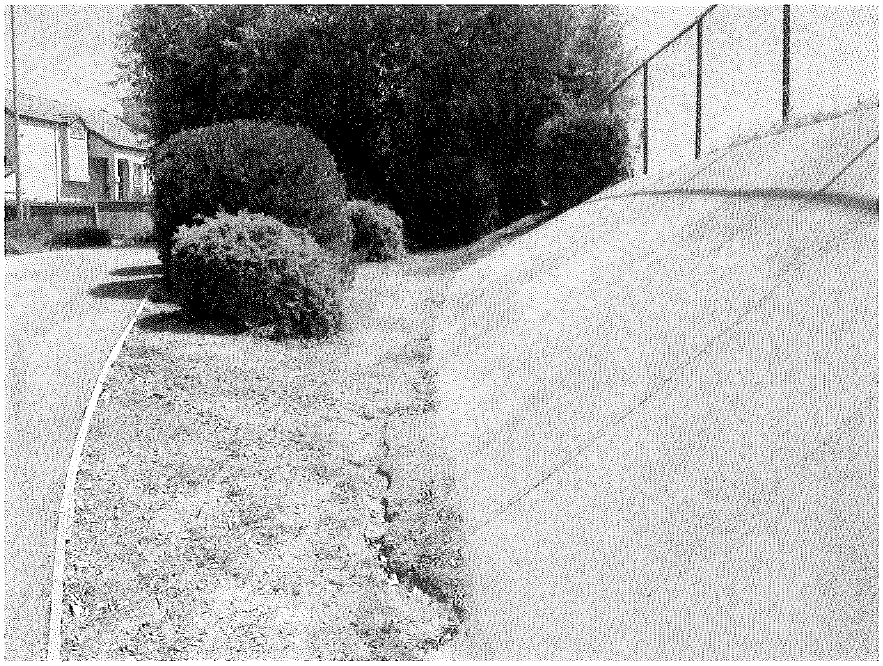
Photograph ID	Approximate Location (Project Station)	Description
053	37+60	North view of outboard retaining wall and electrical tower



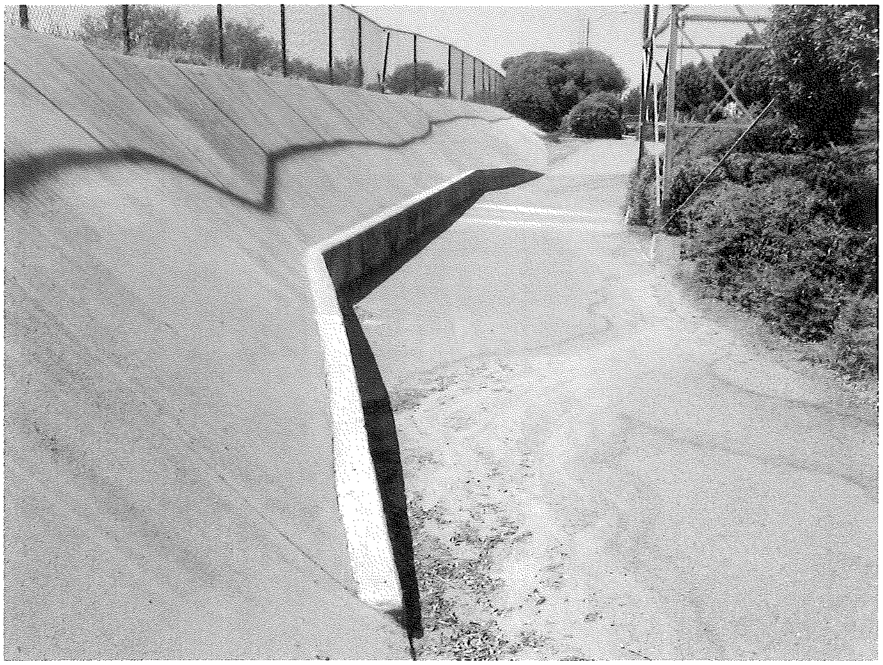
Photograph ID	Approximate Location (Project Station)	Description
054	37+60	South view of inboard levee slope and crest



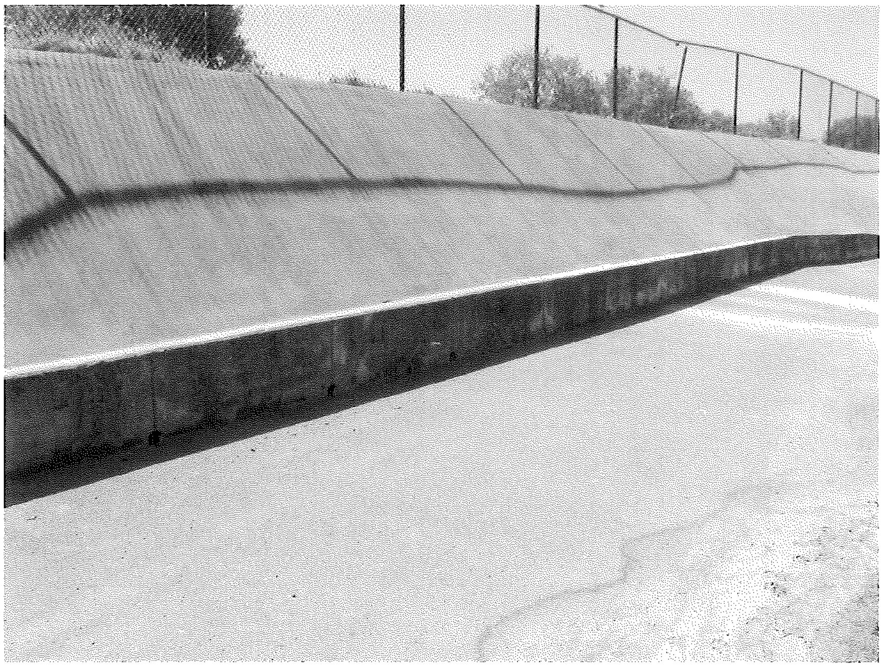
Photograph ID	Approximate Location (Project Station)	Description
055	37+60	South view of inboard levee slope and crest



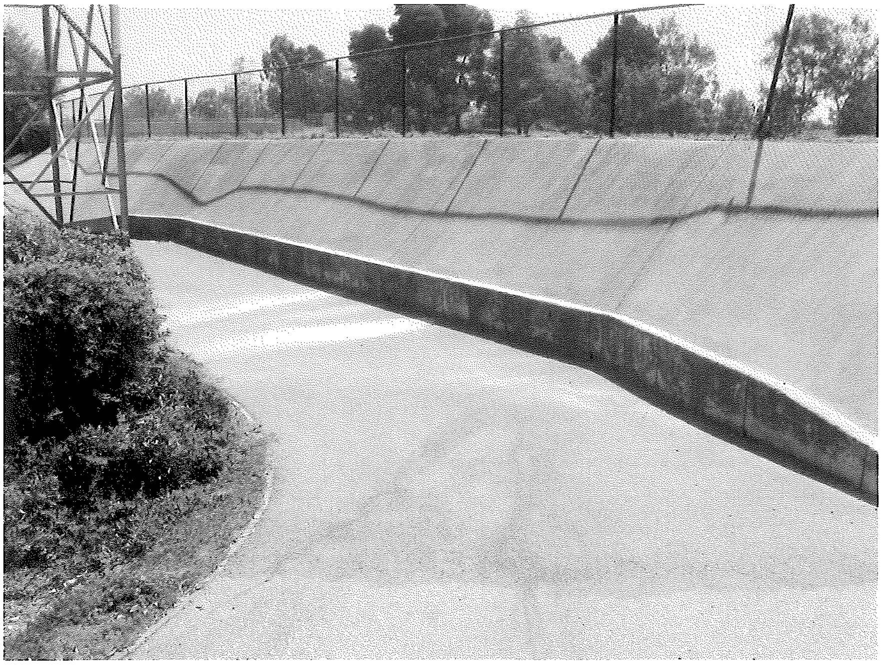
Photograph ID	Approximate Location (Project Station)	Description
056	27+60	South view of outboard levee concrete slope



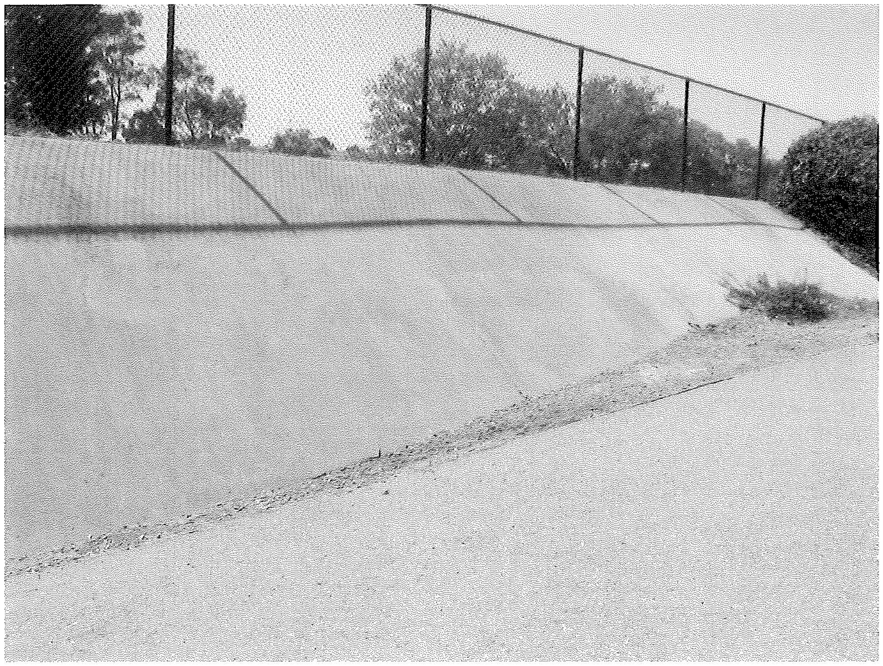
Photograph ID	Approximate Location (Project Station)	Description
057	37+60	North view of outboard levee concrete slope and retaining wall



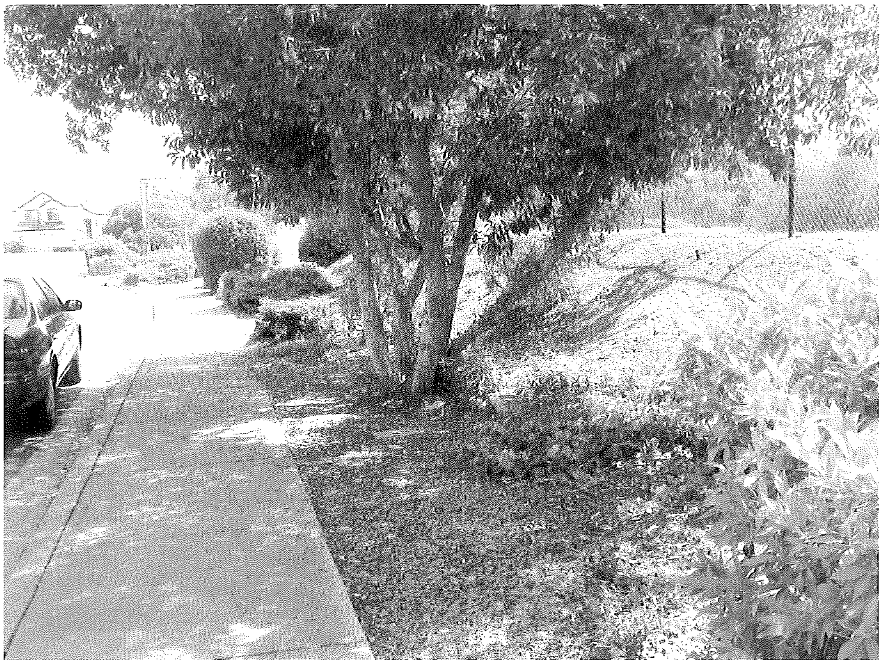
Photograph ID	Approximate Location (Project Station)	Description
058	37+50	Outboard retaining wall and concrete slope



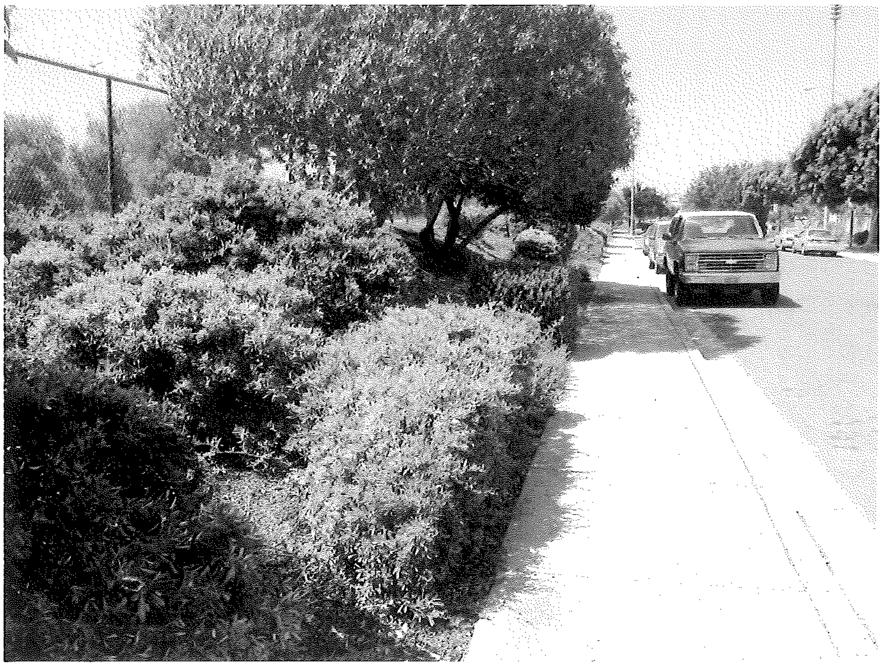
Photograph ID	Approximate Location (Project Station)	Description
059	36+60	South view of outboard levee concrete slope and retaining wall



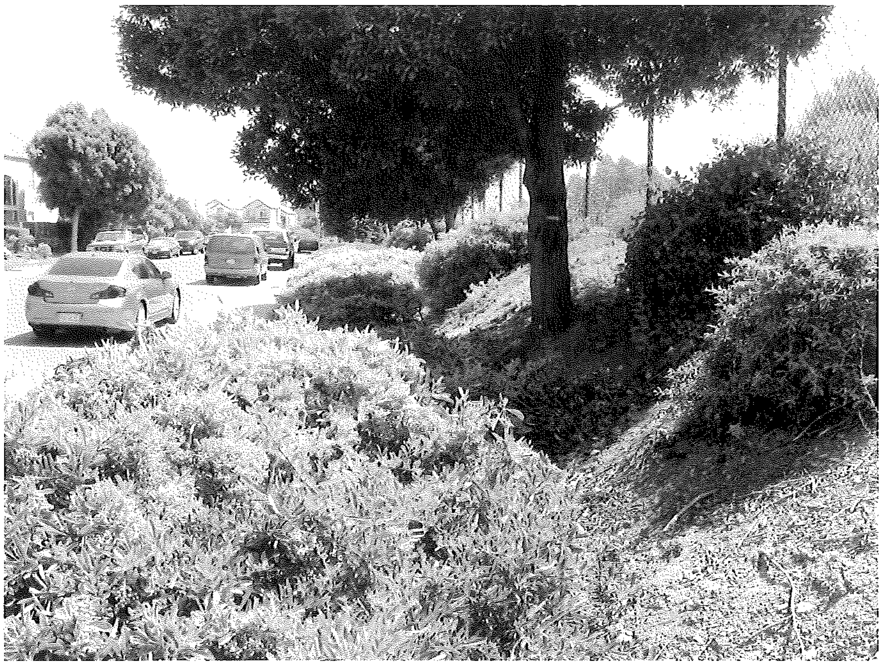
Photograph ID	Approximate Location (Project Station)	Description
060	36+60	North view of outboard concrete slope



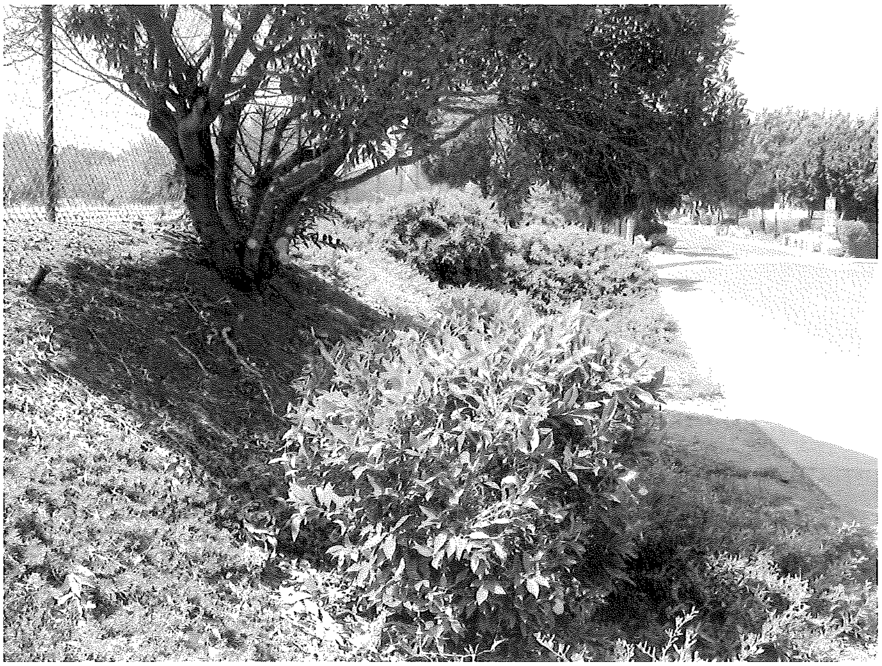
Photograph ID	Approximate Location (Project Station)	Description
061	35+00	South view of outboard levee slope



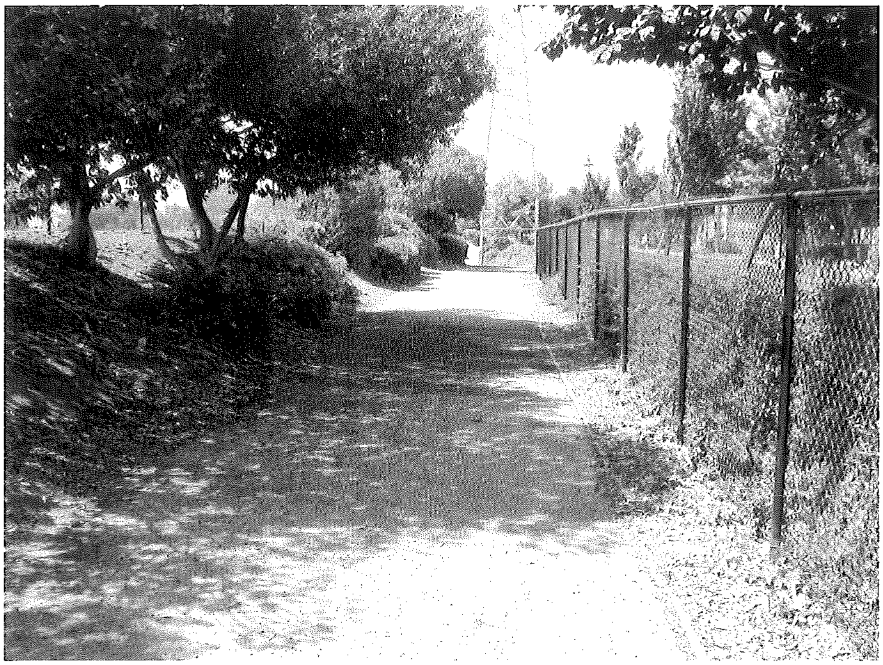
Photograph ID	Approximate Location (Project Station)	Description
062	35+00	North view of outboard levee slope



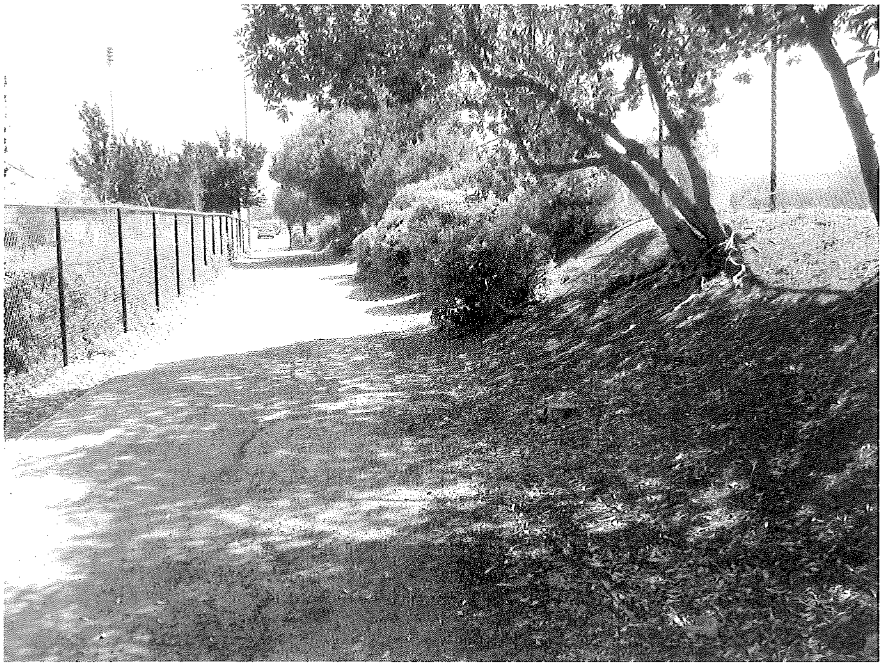
Photograph ID	Approximate Location (Project Station)	Description
063	33+00	South view of outboard levee slope



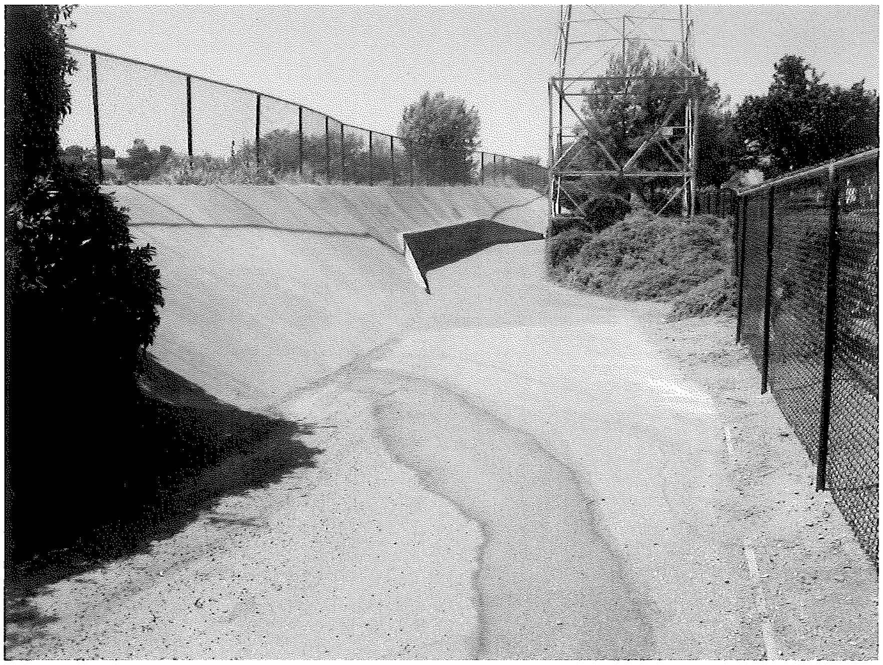
Photograph ID	Approximate Location (Project Station)	Description
064	33+00	North view of outboard levee slope



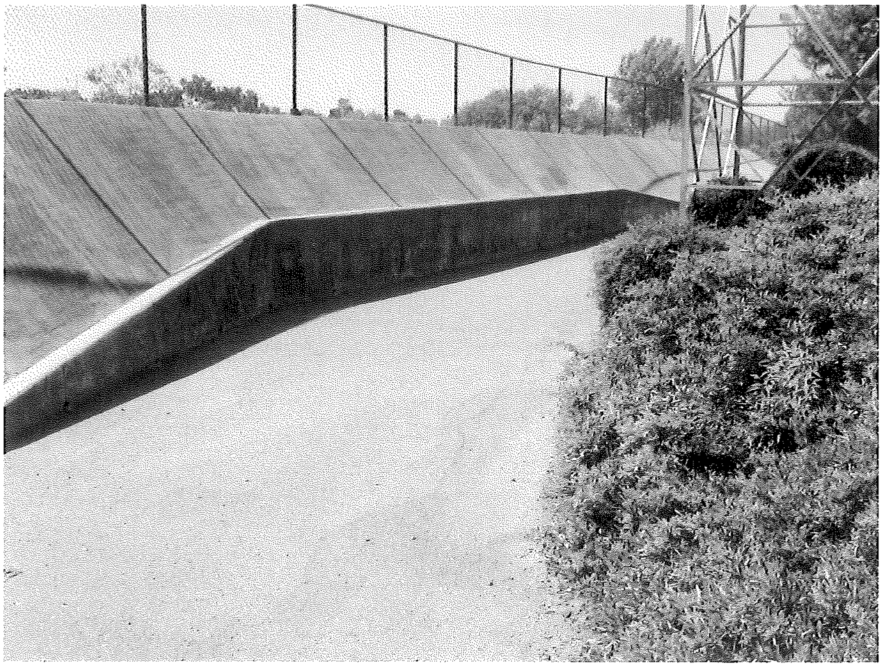
Photograph ID	Approximate Location (Project Station)	Description
065	31+90	North view of outboard levee slope



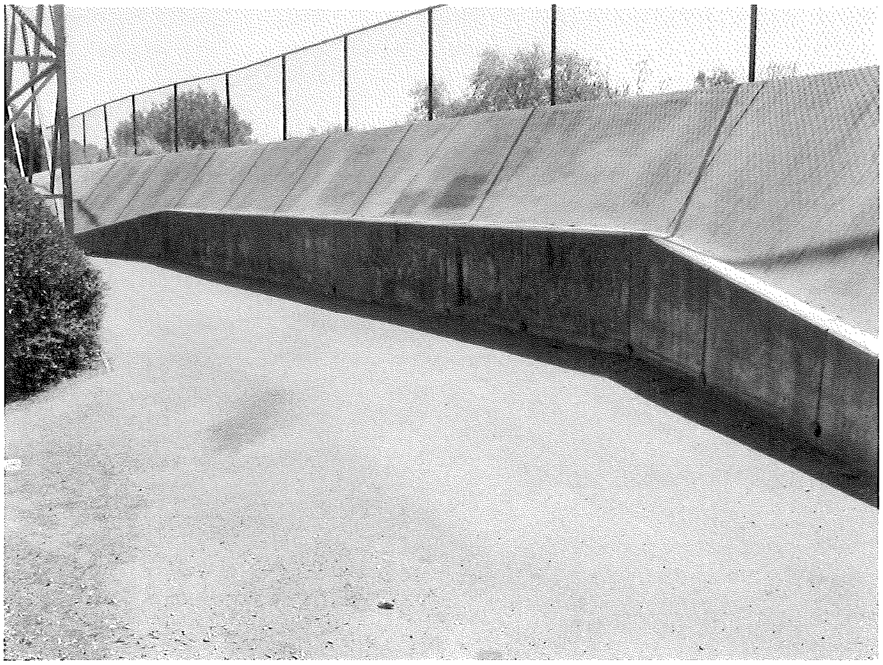
Photograph ID	Approximate Location (Project Station)	Description
066	30+20	South view of outboard levee slope



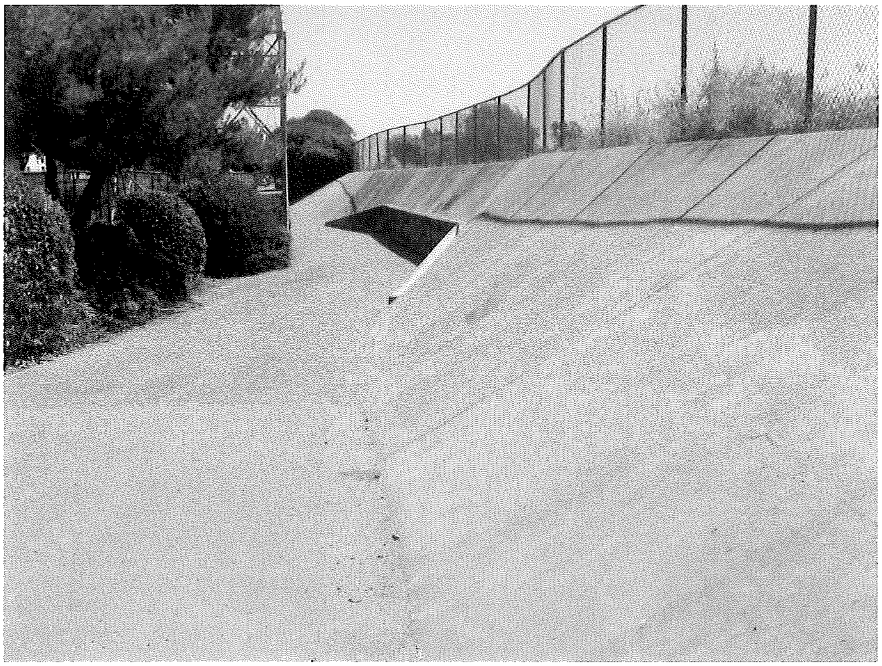
Photograph ID	Approximate Location (Project Station)	Description
067	30+20	North view of outboard levee concrete slope and retaining wall



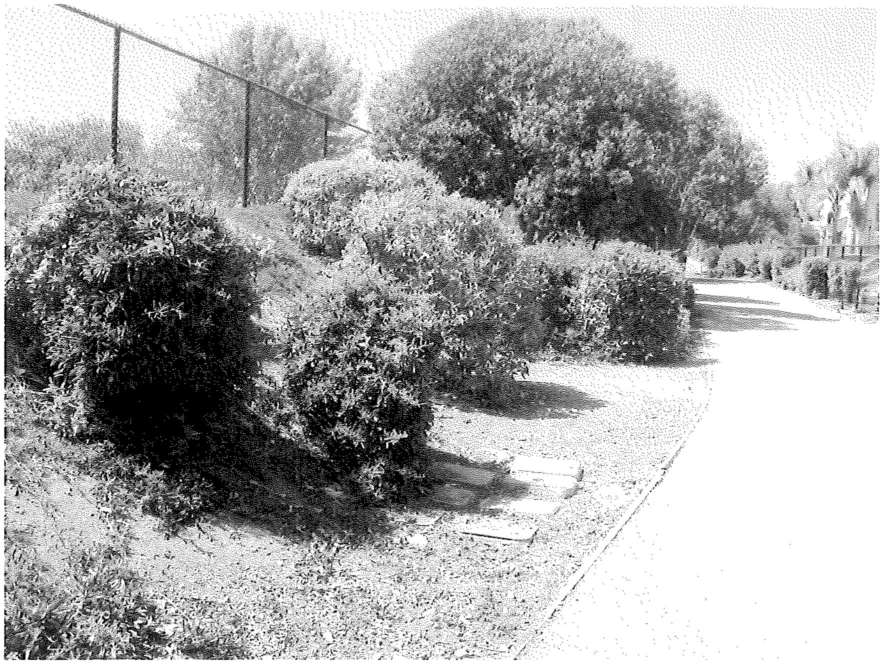
Photograph ID	Approximate Location (Project Station)	Description
068	29+80	North view of outboard levee concrete slope and retaining wall



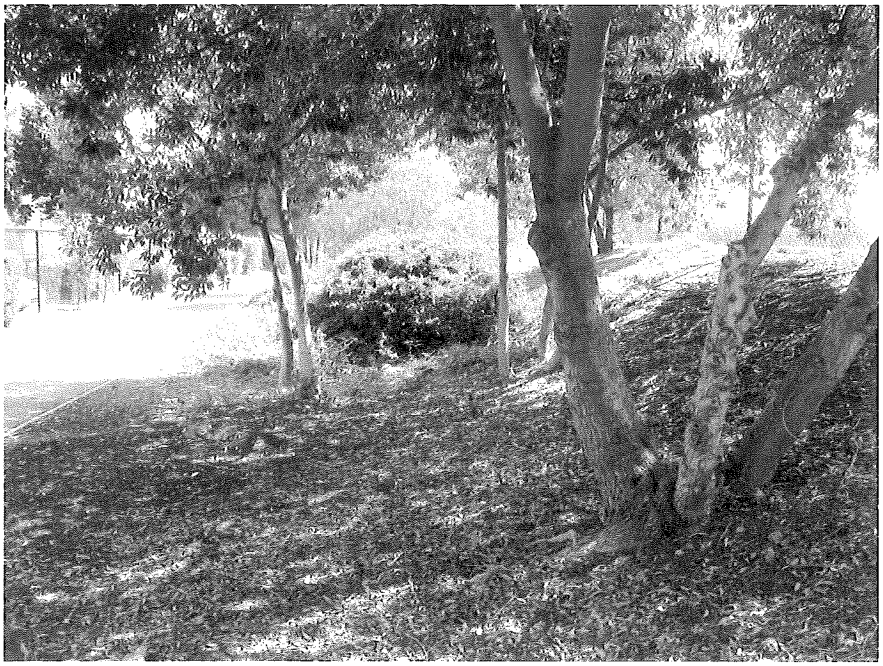
Photograph ID	Approximate Location (Project Station)	Description
069	28+70	South view of outboard levee concrete slope and retaining wall



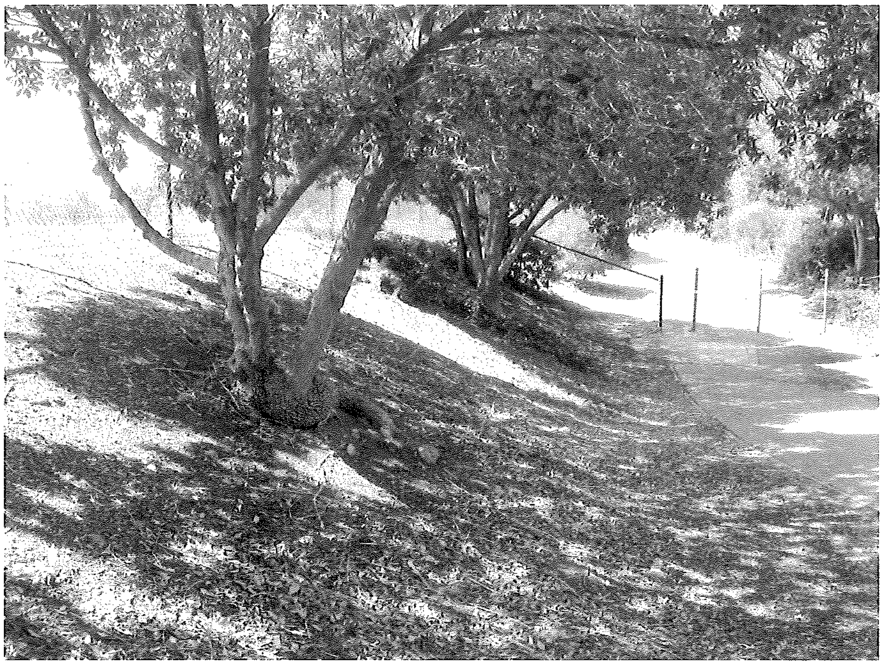
Photograph ID	Approximate Location (Project Station)	Description
070	28+20	South view of levee outboard concrete slope



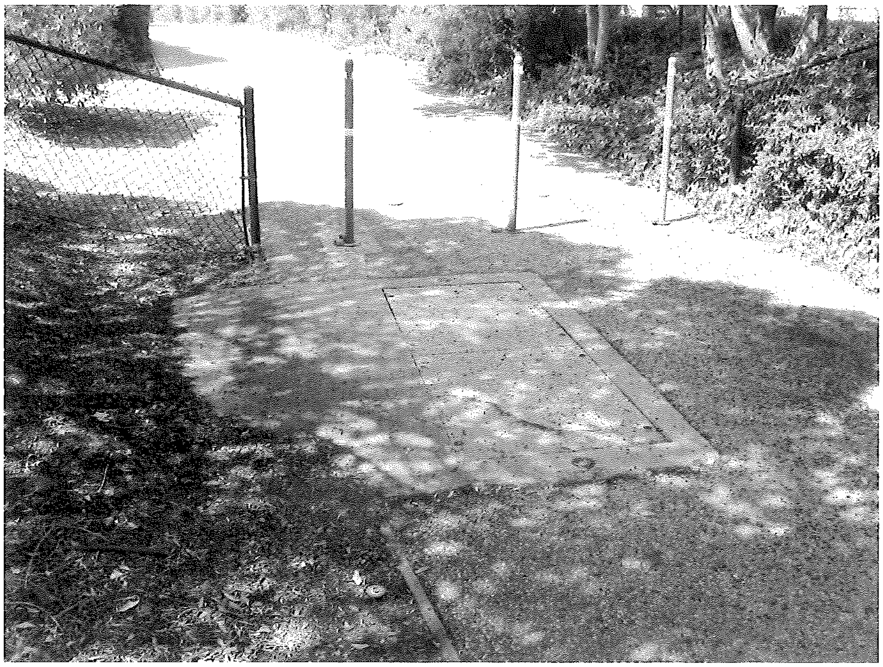
Photograph ID	Approximate Location (Project Station)	Description
071	28+20	North view of outboard levee slope



Photograph ID	Approximate Location (Project Station)	Description
072	26+50	South view of outboard levee slope



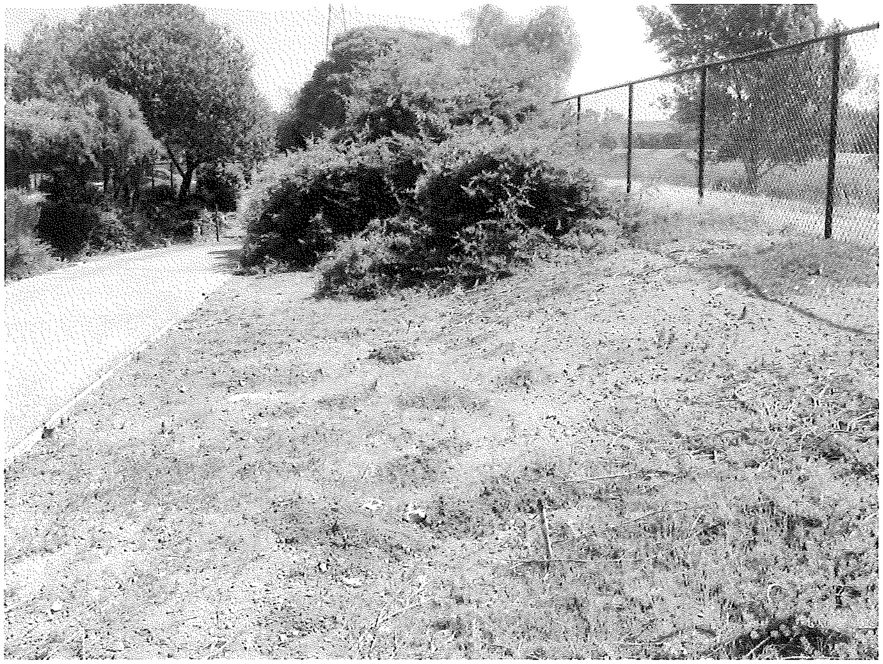
Photograph ID	Approximate Location (Project Station)	Description
073	26+50	North view of outboard levee slope



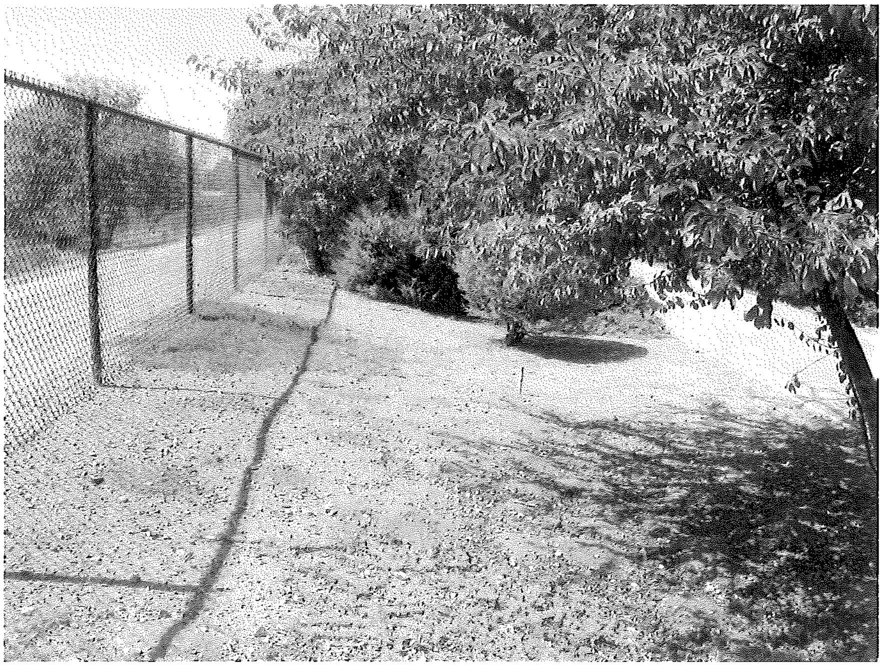
Photograph ID	Approximate Location (Project Station)	Description
074	26+00	Storm drain vault

Photograph ID	Approximate Location (Project Station)	Description
075	26+00	Sanitary sewer manhole

Photograph ID	Approximate Location (Project Station)	Description
076	25+70	South view of storm drain vault and sanitary sewer manhole



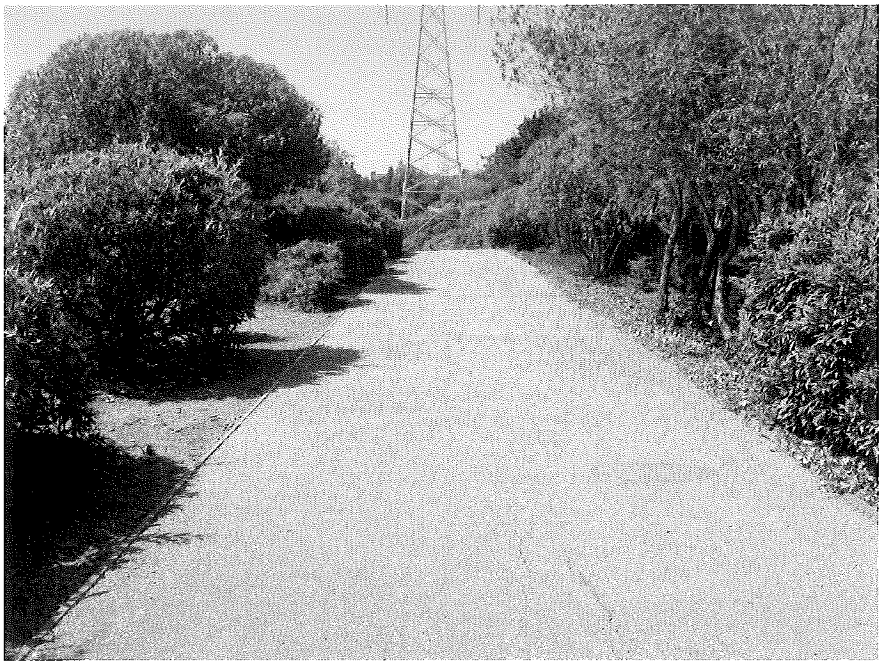
Photograph ID	Approximate Location (Project Station)	Description
077	25+00	South view of outboard levee slope



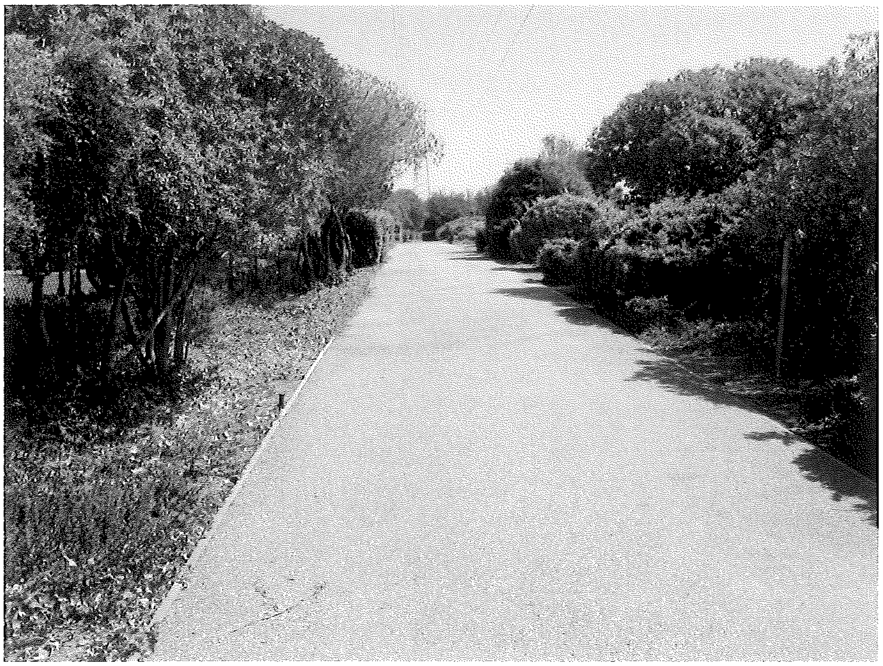
Photograph ID	Approximate Location (Project Station)	Description
078	25+00	North view of outboard levee slope

Photograph ID	Approximate Location (Project Station)	Description
079	25+00	North view of outboard pedestrian trail

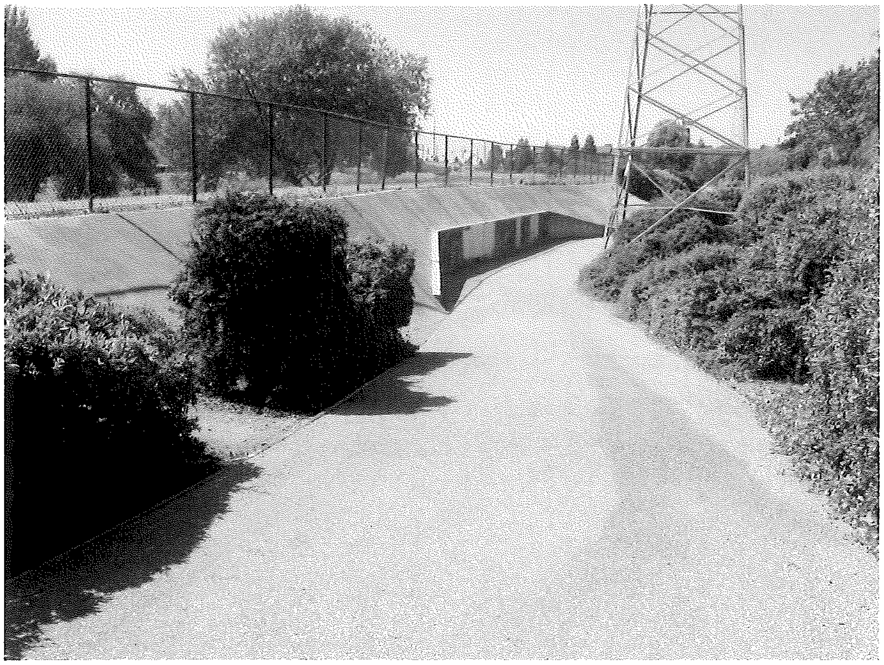
Photograph ID	Approximate Location (Project Station)	Description
080	23+50	South view of outboard levee slope and pedestrian trail



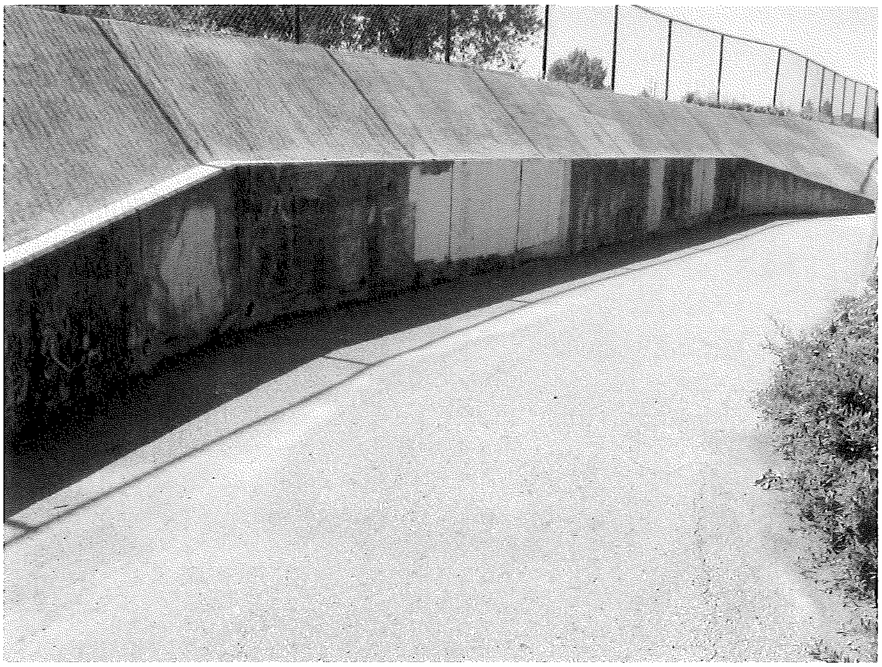
Photograph ID	Approximate Location (Project Station)	Description
081	23+50	North view of outboard levee slope and pedestrian trail



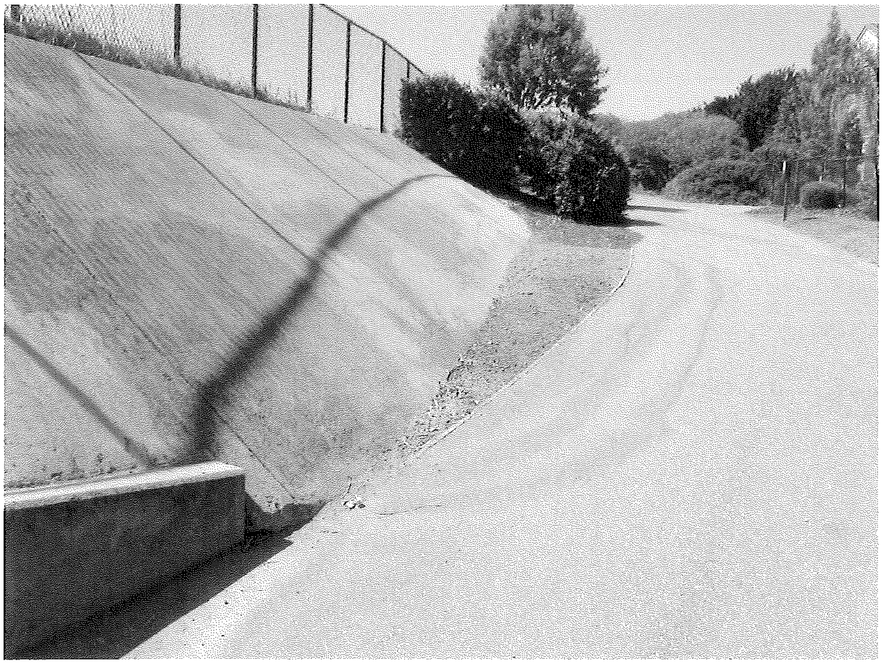
Photograph ID	Approximate Location (Project Station)	Description
082	22+50	South view of outboard levee pedestrian road



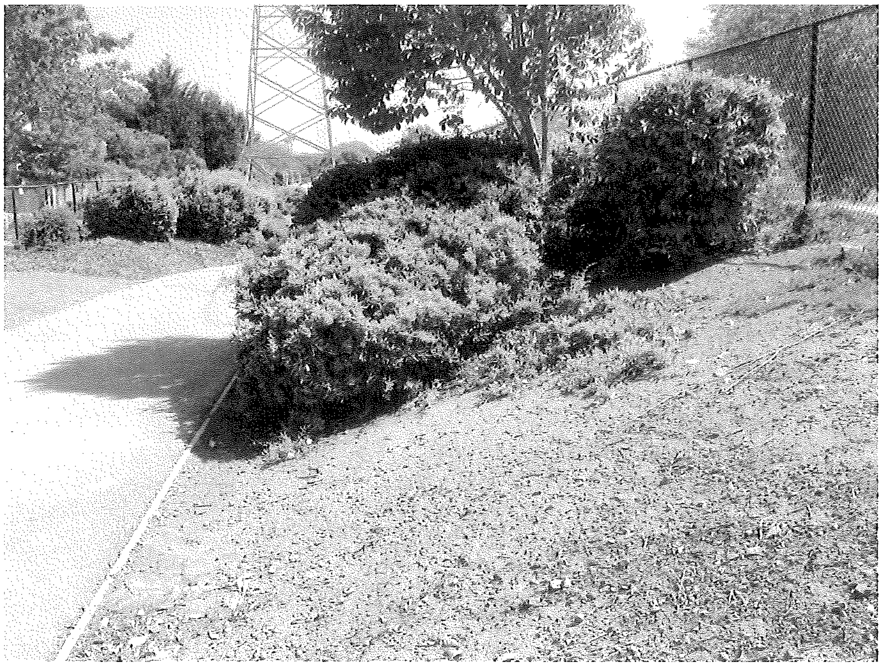
Photograph ID	Approximate Location (Project Station)	Description
083	22+50	North view of outboard levee concrete slope



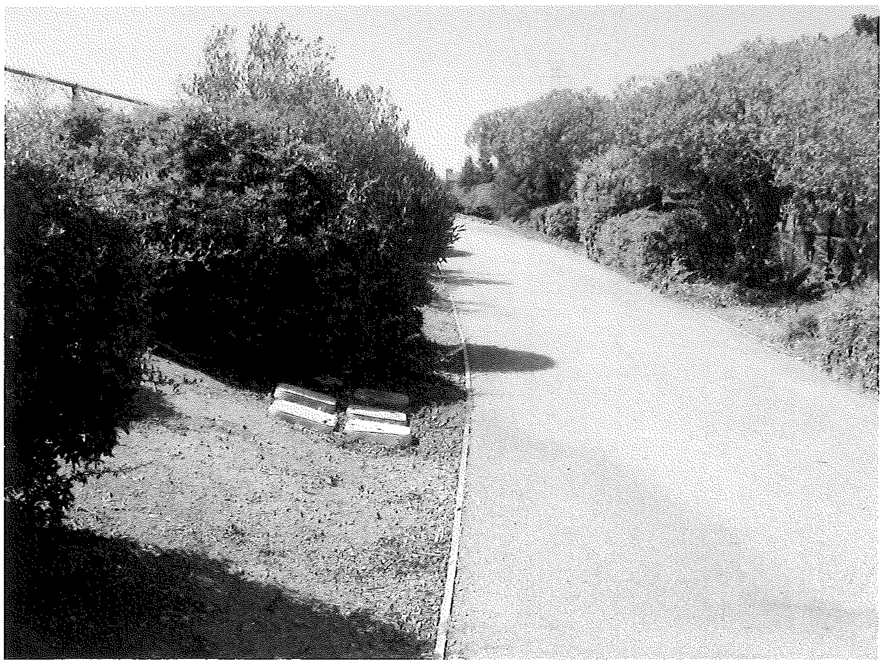
Photograph ID	Approximate Location (Project Station)	Description
084	21+80	North view of outboard levee concrete wall and slope



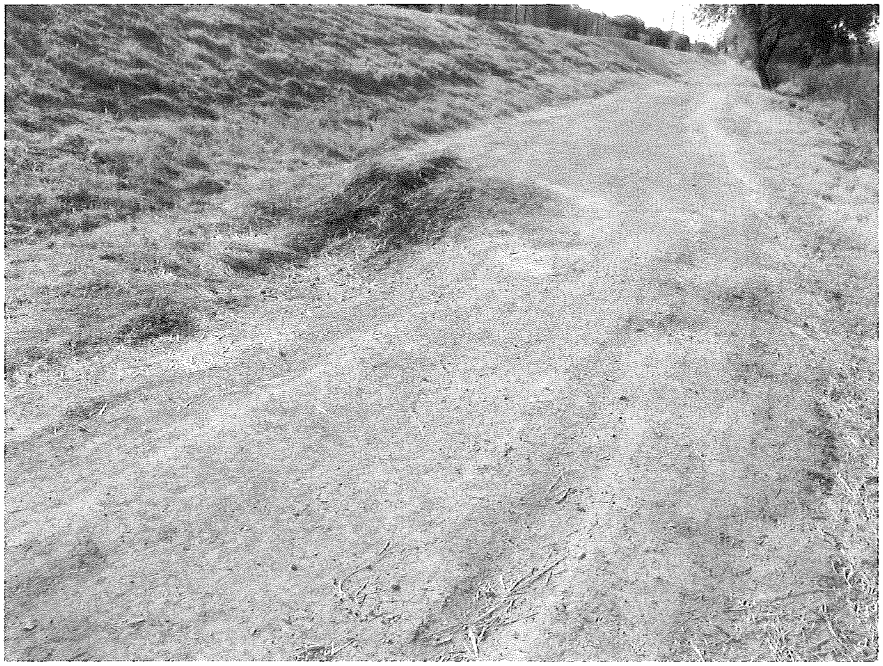
Photograph ID	Approximate Location (Project Station)	Description
087	20+90	North view of outboard levee concrete wall and slope



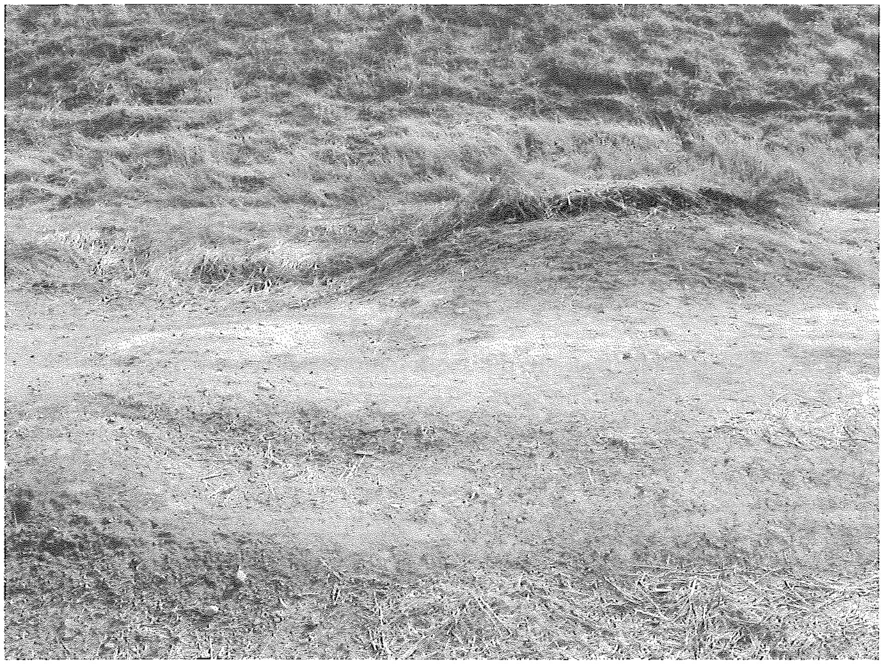
Photograph ID	Approximate Location (Project Station)	Description
088	19+50	South view of outboard levee slope



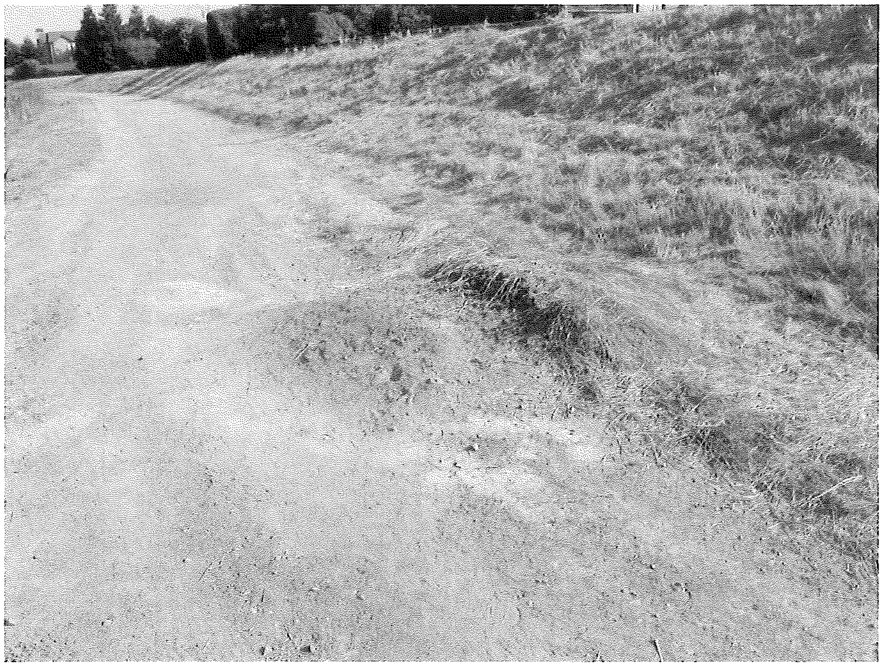
Photograph ID	Approximate Location (Project Station)	Description
089	19+50	North view of outboard levee slope



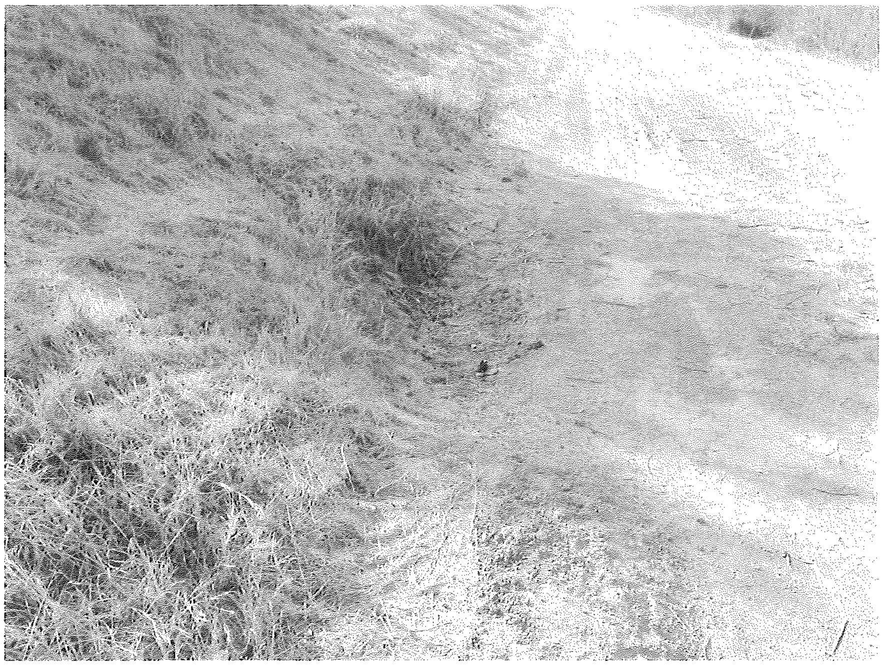
Photograph ID	Approximate Location (Project Station)	Description
090	19+30	Ruts and bump in lower maintenance road



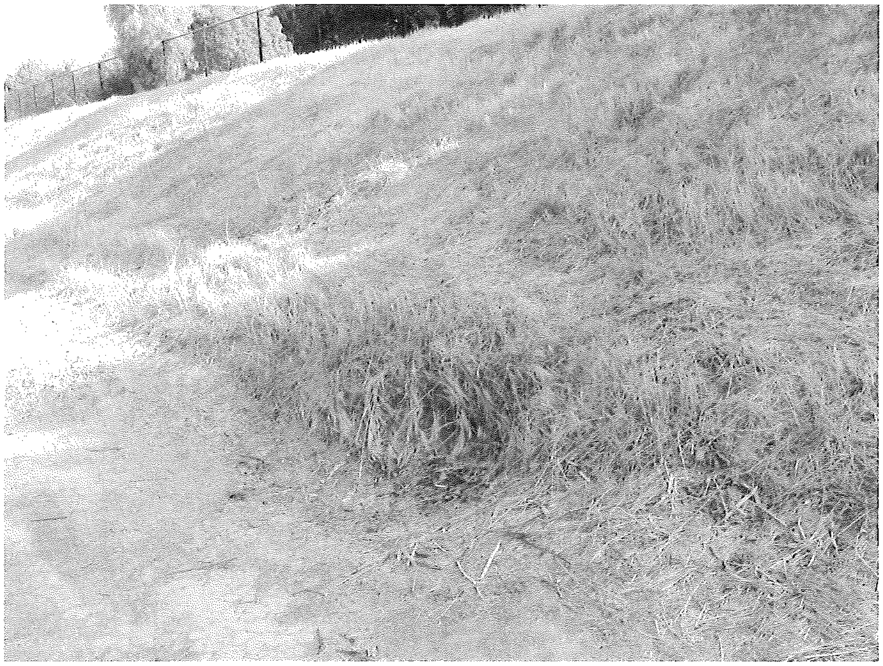
Photograph ID	Approximate Location (Project Station)	Description
091	19+30	Ruts and bump in lower maintenance road



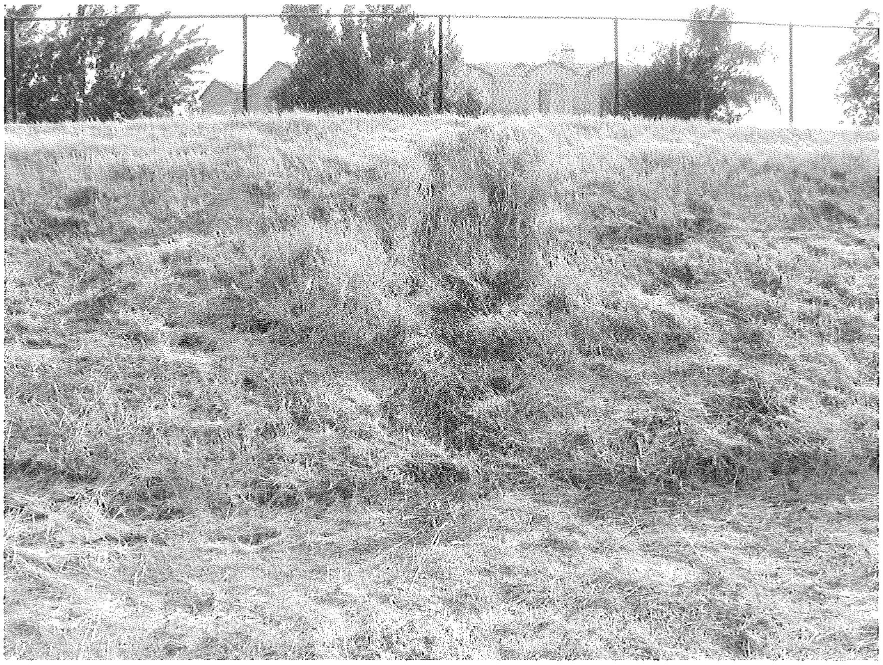
Photograph ID	Approximate Location (Project Station)	Description
092	19+30	Ruts and bump in lower maintenance road



Photograph ID	Approximate Location (Project Station)	Description
093	26+30	Rutting/oversteepening at levee toe

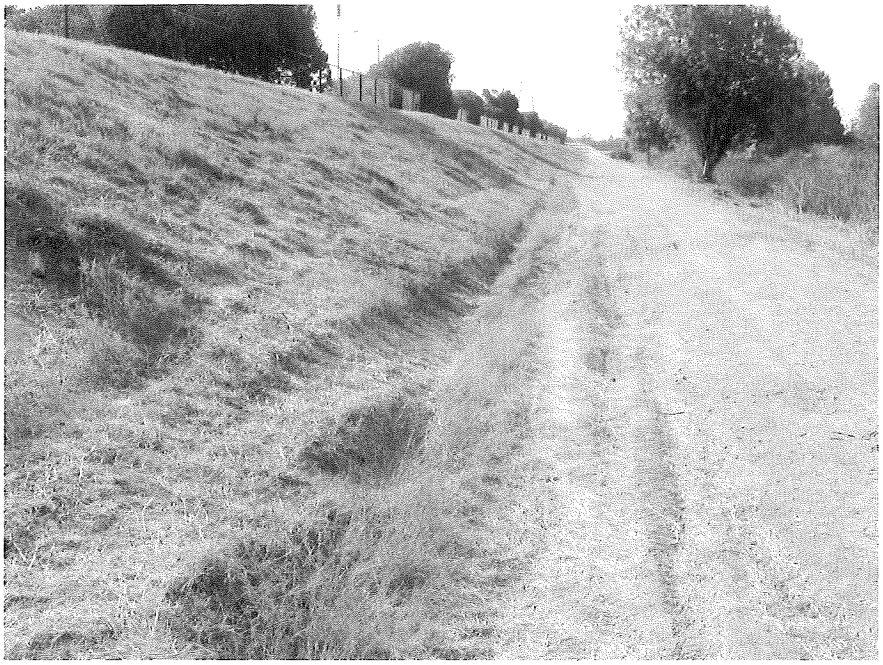


Photograph ID	Approximate Location (Project Station)	Description
094	26+30	Rutting/oversteepening at levee toe

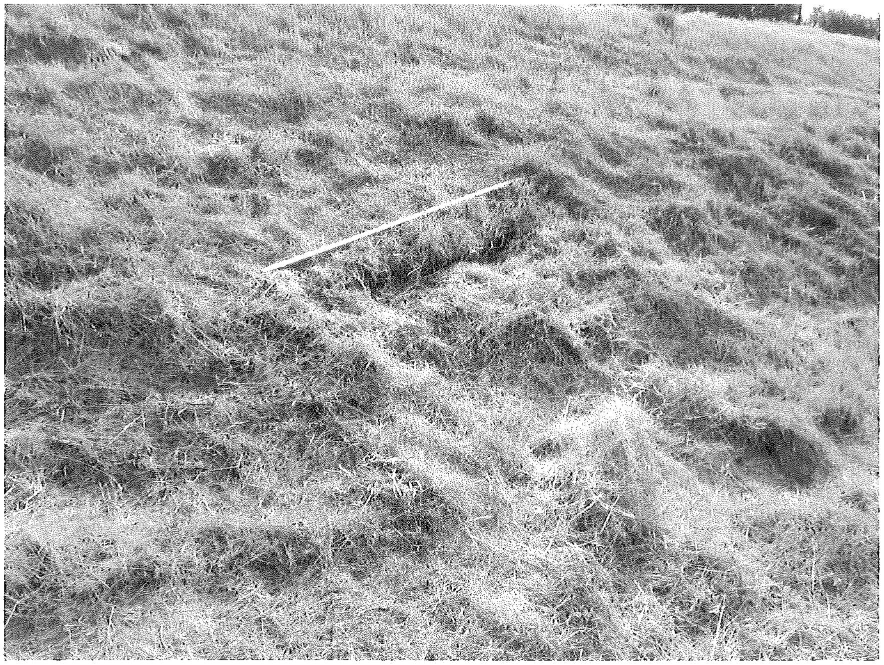


Photograph ID	Approximate Location (Project Station)	Description
095	28+00	Minor erosion gully on inboard slope

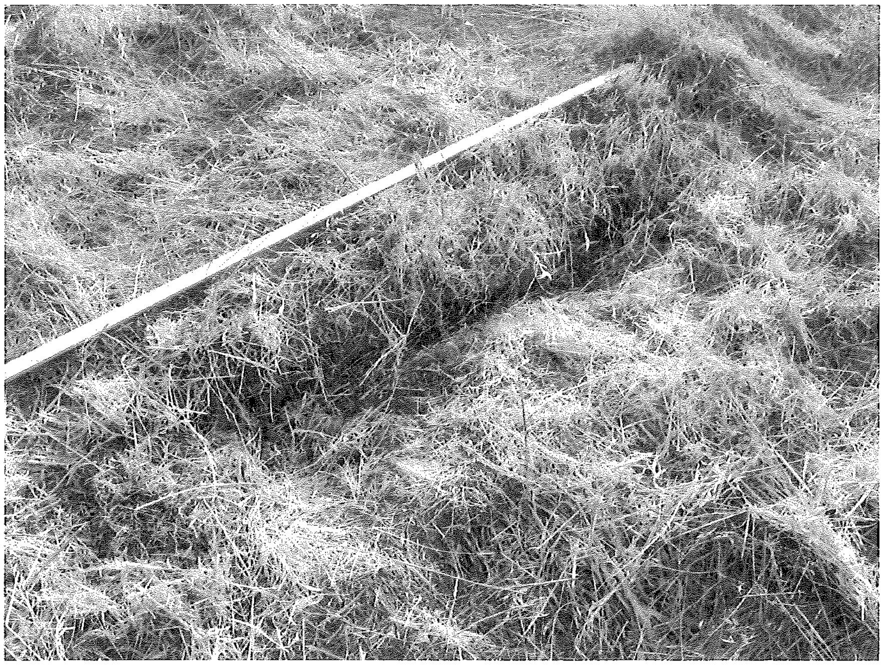
Photograph ID	Approximate Location (Project Station)	Description
096	28+00	Minor erosion gully on inboard slope



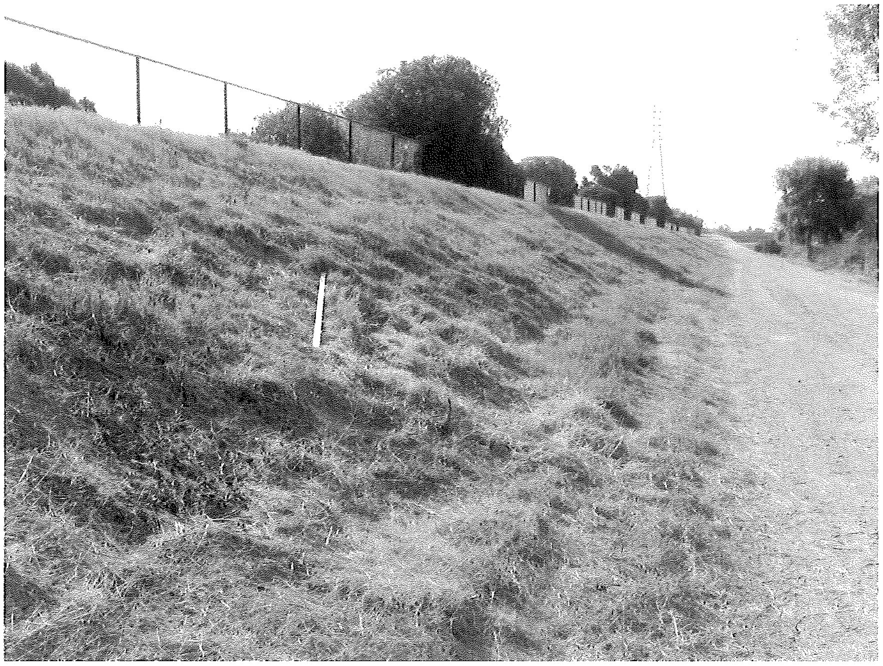
Photograph ID	Approximate Location (Project Station)	Description
097	29+50	Rutting at toe of inboard slope



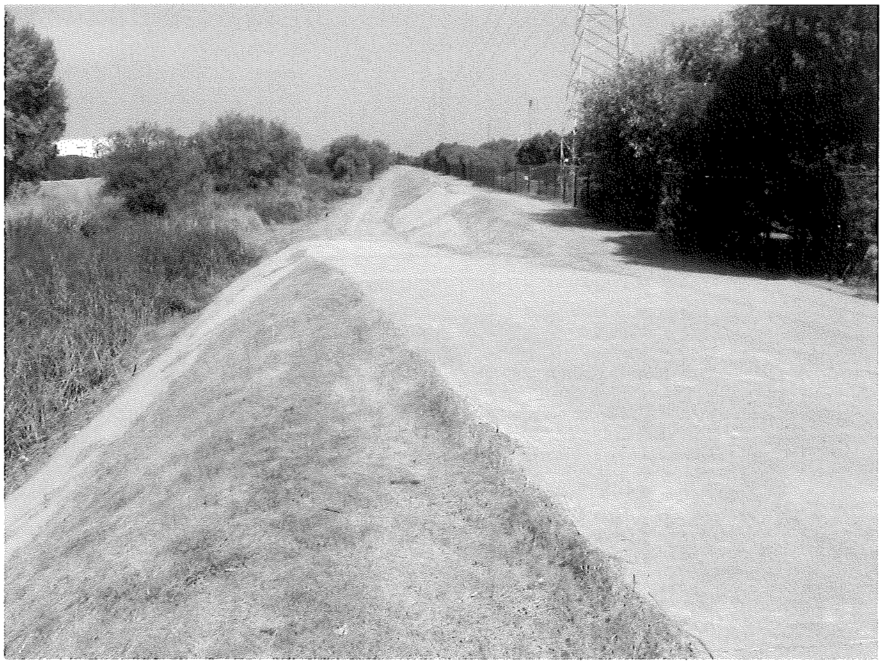
Photograph ID	Approximate Location (Project Station)	Description
098	30+50	Ground crack on lower inboard slope



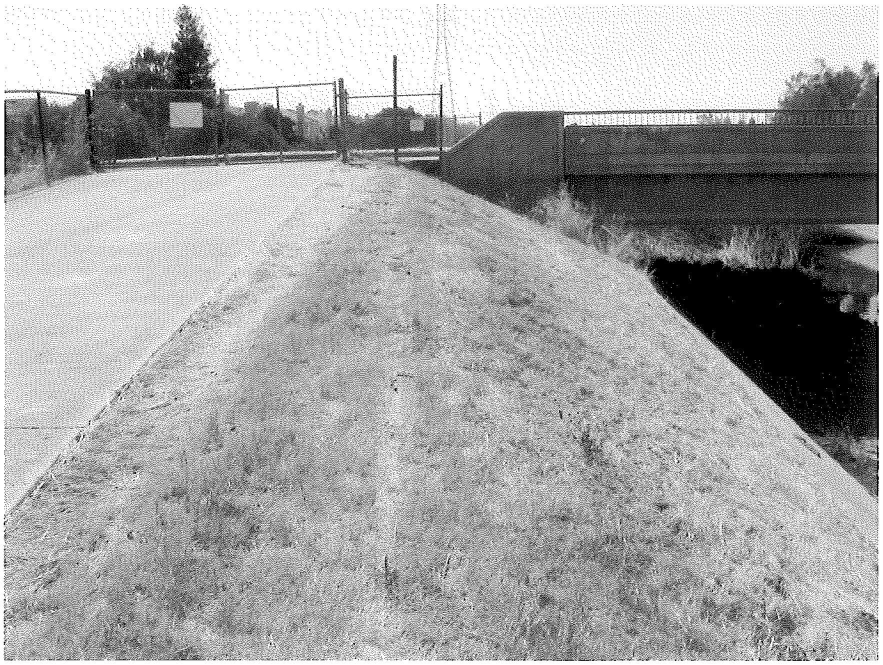
Photograph ID	Approximate Location (Project Station)	Description
099	30+50	Ground crack on lower inboard slope



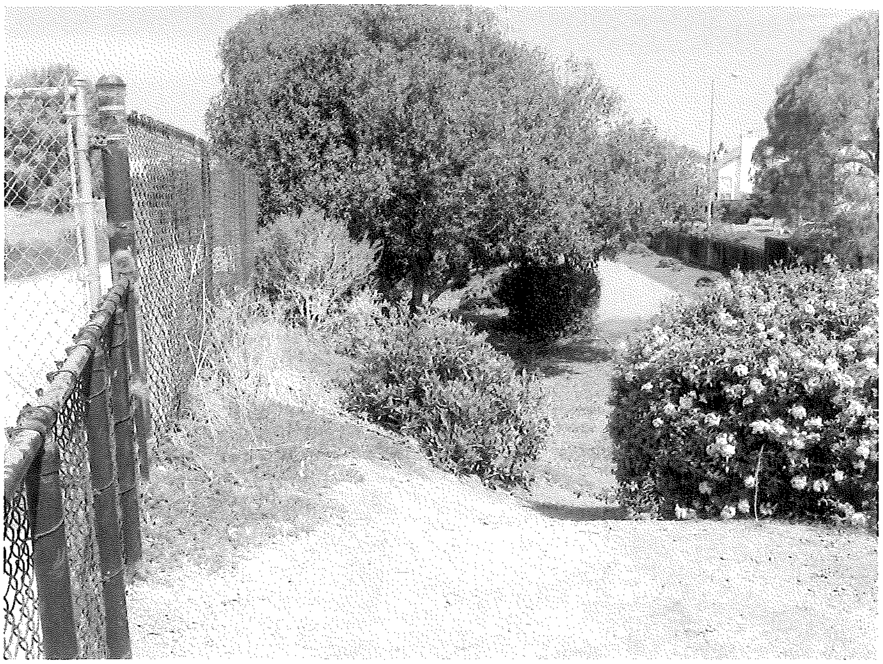
Photograph ID	Approximate Location (Project Station)	Description
100	30+50	Ground crack on lower inboard slope



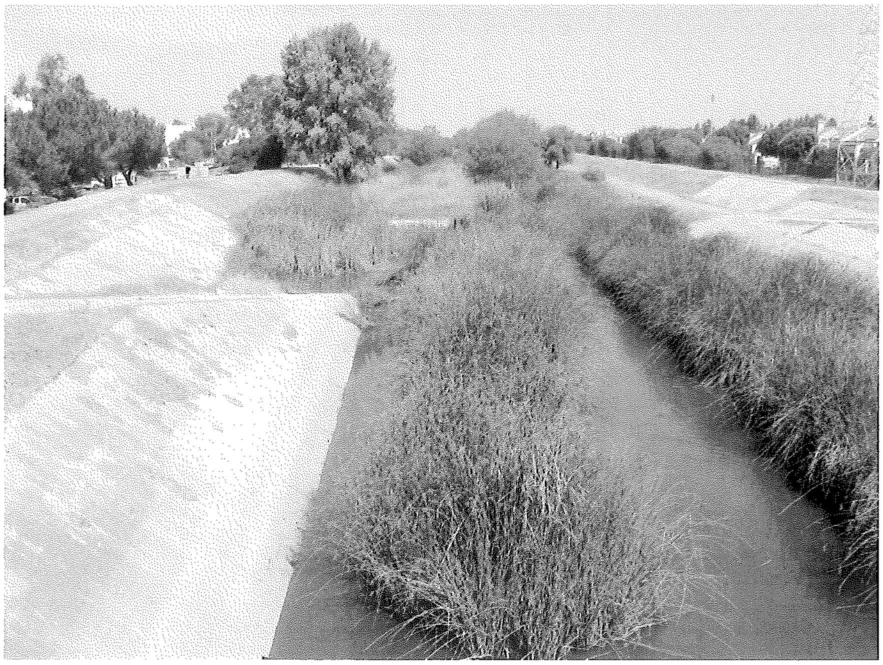
Photograph ID	Approximate Location (Project Station)	Description
101	39+20	North view of inboard levee slope



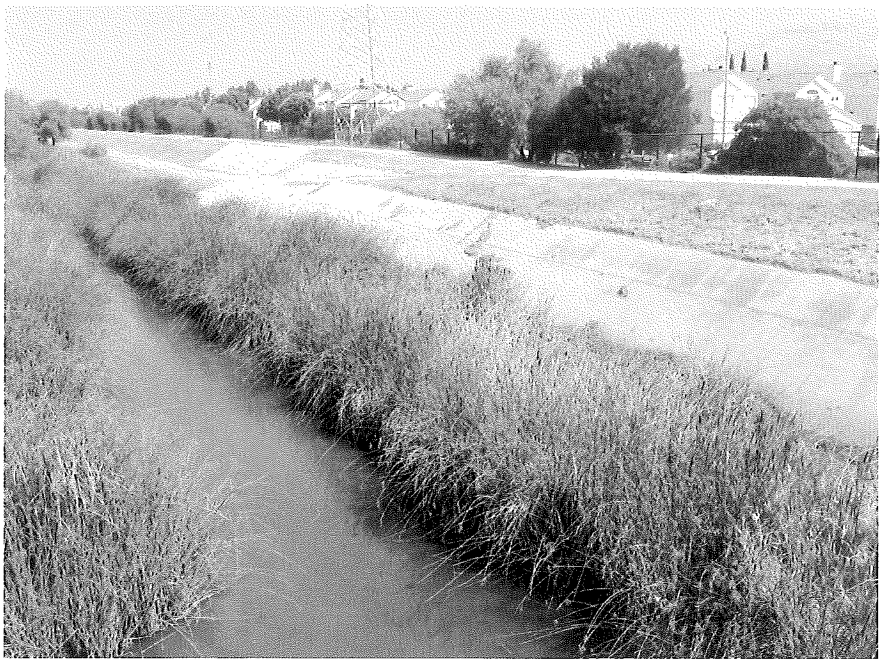
Photograph ID	Approximate Location (Project Station)	Description
102	39+20	South view of inboard levee slope



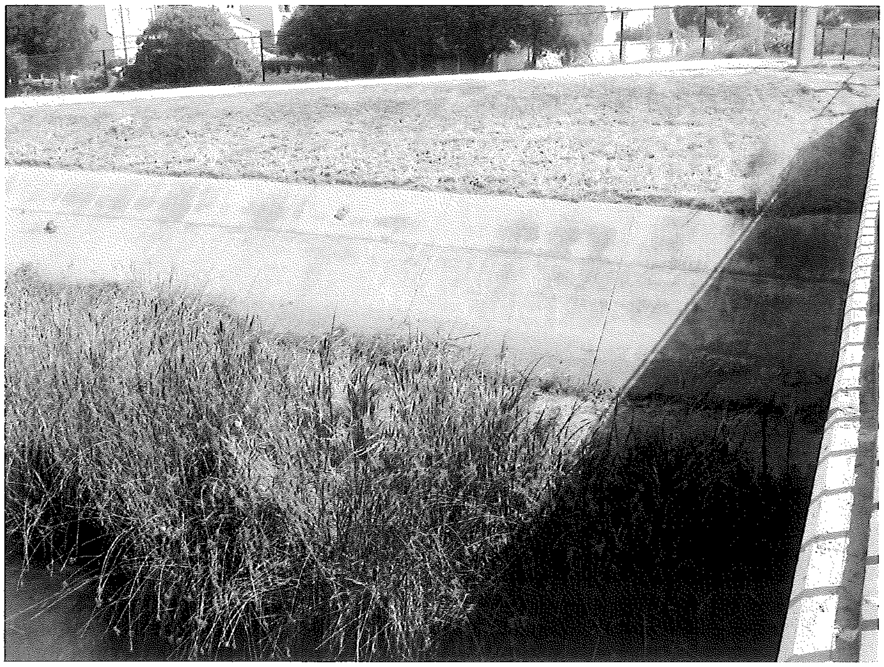
Photograph ID	Approximate Location (Project Station)	Description
103	40+10	North view of outboard slope from Milmont Avenue Bridge



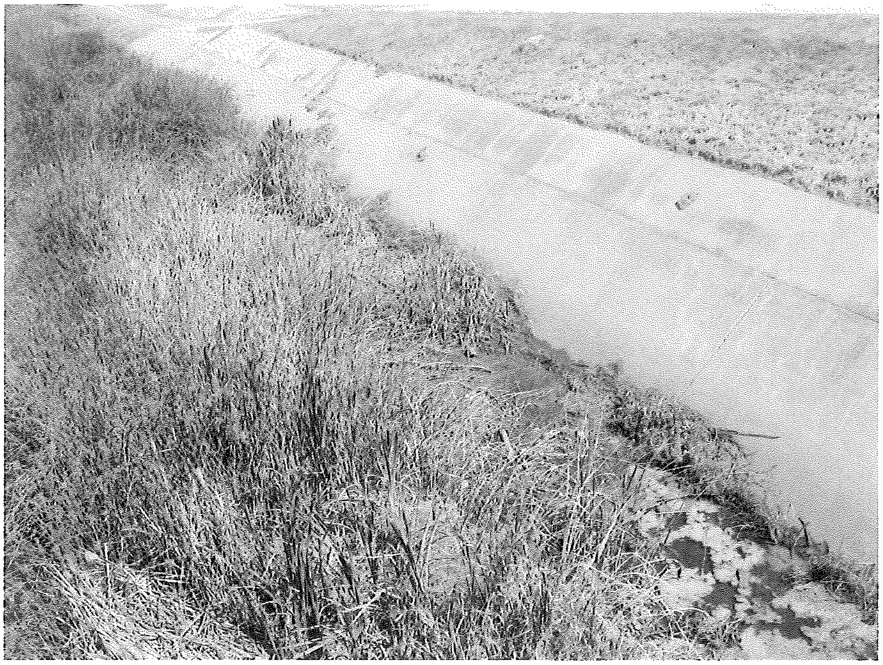
Photograph ID	Approximate Location (Project Station)	Description
104	40+00	North view of channel and slopes from Milmont Avenue Bridge



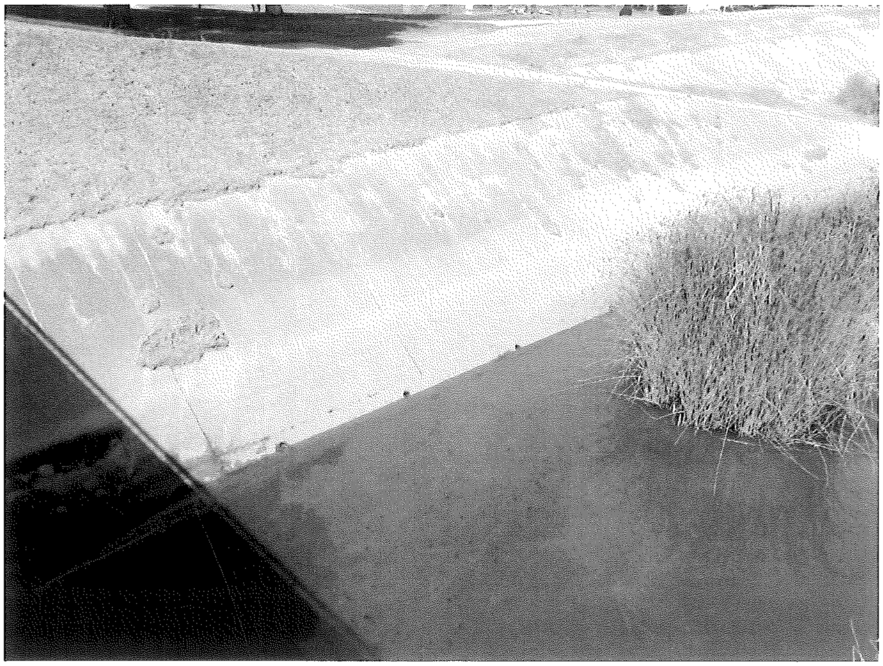
Photograph ID	Approximate Location (Project Station)	Description
105	40+00	North view of channel and east slope from Milmont Avenue Bridge



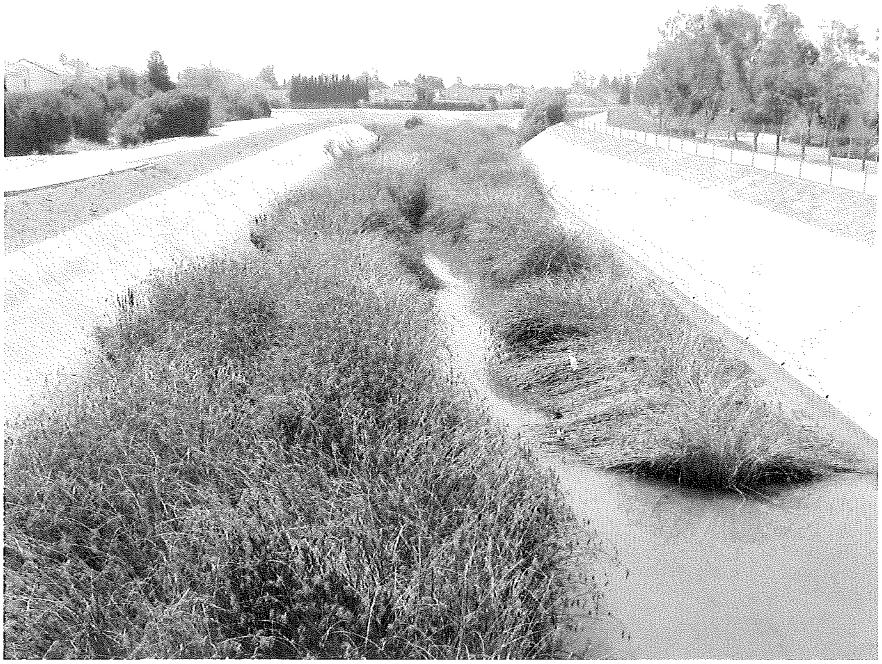
Photograph ID	Approximate Location (Project Station)	Description
106	40+00	North view of channel and east slope from Milmont Avenue Bridge



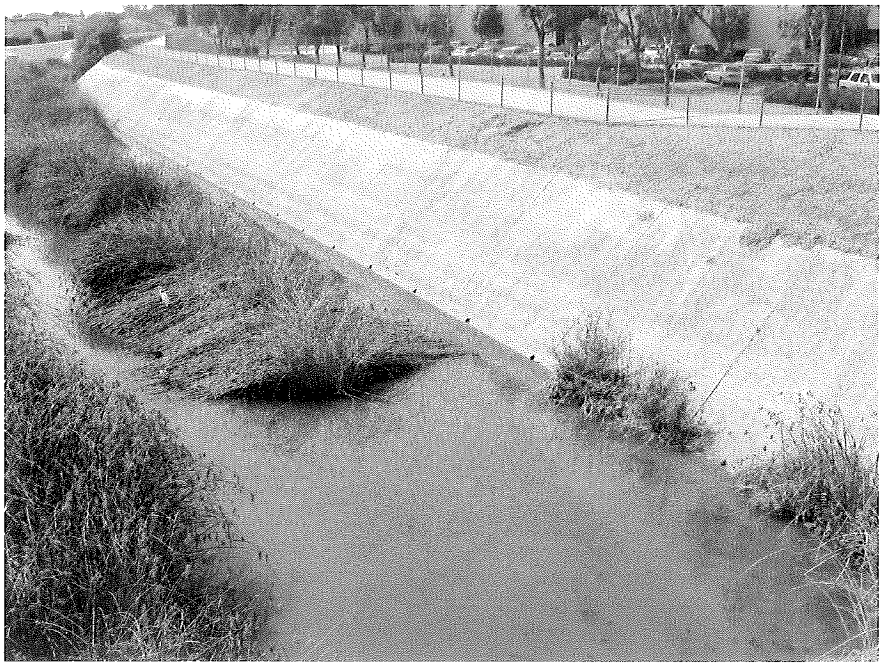
Photograph ID	Approximate Location (Project Station)	Description
107	40+00	North view of channel and east slope from Milmont Avenue Bridge



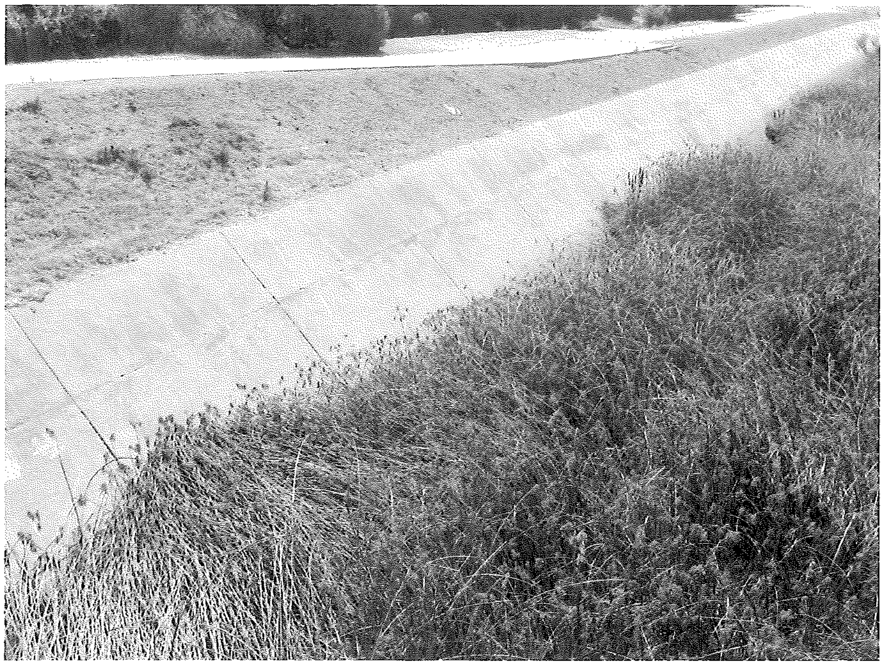
Photograph ID	Approximate Location (Project Station)	Description
108	40+00	North view of channel and west slope from Milmont Avenue Bridge



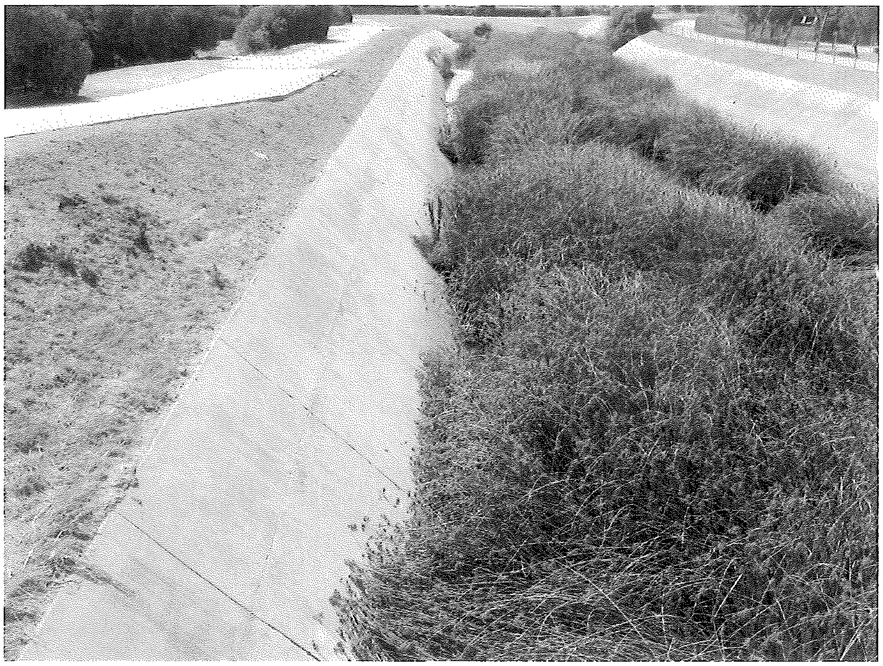
Photograph ID	Approximate Location (Project Station)	Description
109	40+80	South view of channel and slopes from Milmont Avenue Bridge



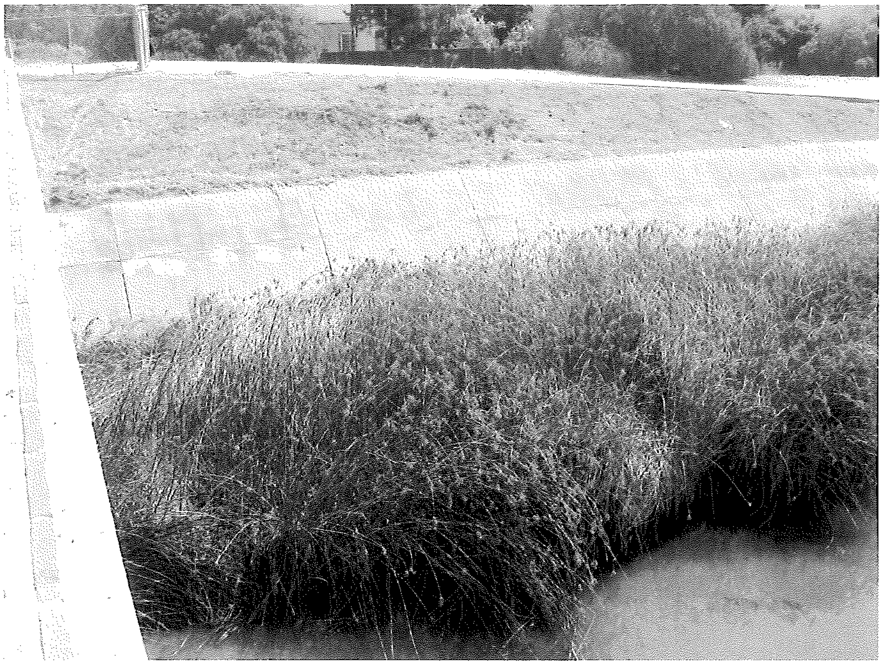
Photograph ID	Approximate Location (Project Station)	Description
110	40+80	South view of channel and west slope from Milmont Avenue Bridge



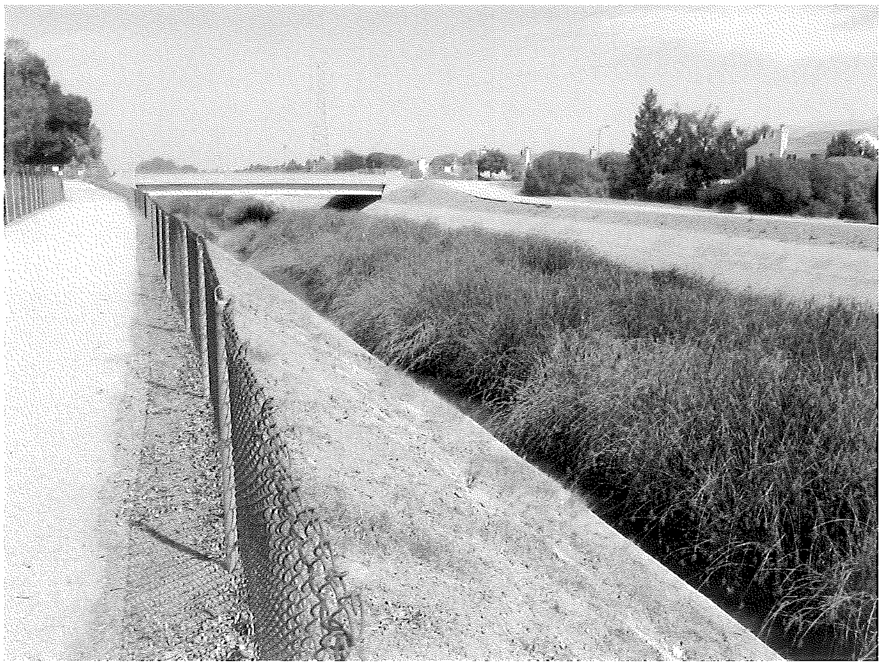
Photograph ID	Approximate Location (Project Station)	Description
111	40+80	South view of channel and east slope from Milmont Avenue Bridge



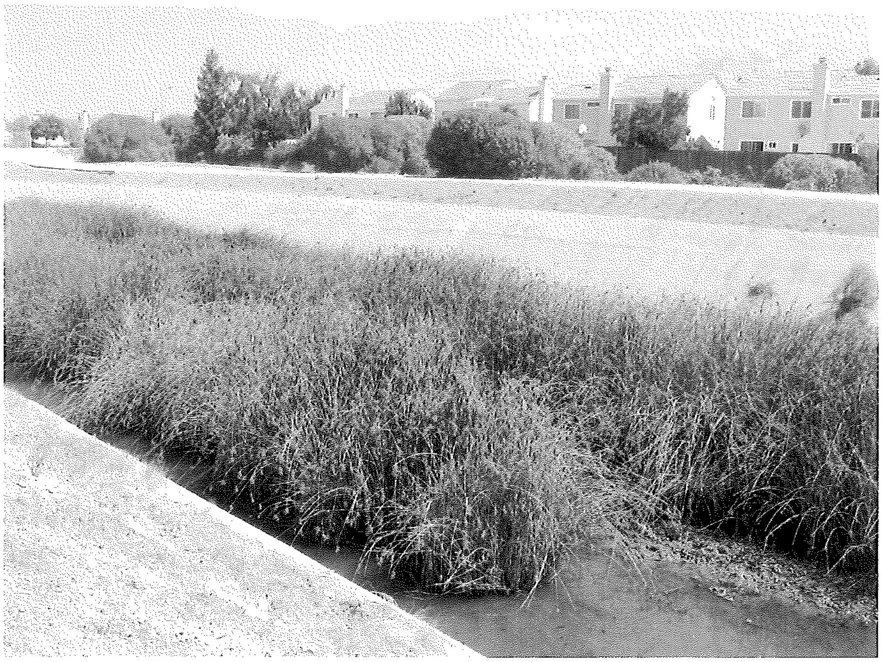
Photograph ID	Approximate Location (Project Station)	Description
112	40+80	South view of channel and east slope from Milmont Avenue Bridge



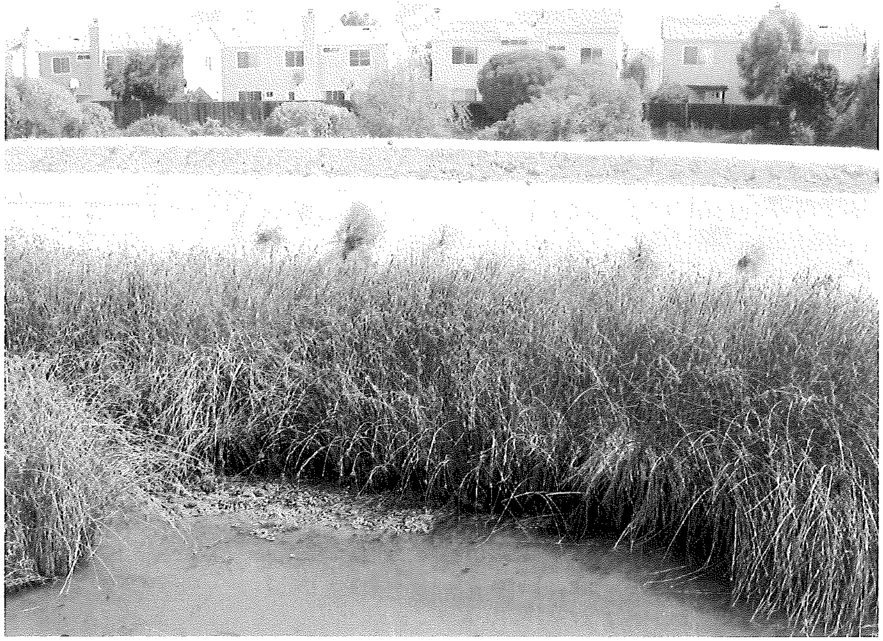
Photograph ID	Approximate Location (Project Station)	Description
113	40+80	South view of channel and east slope from Milmont Avenue Bridge



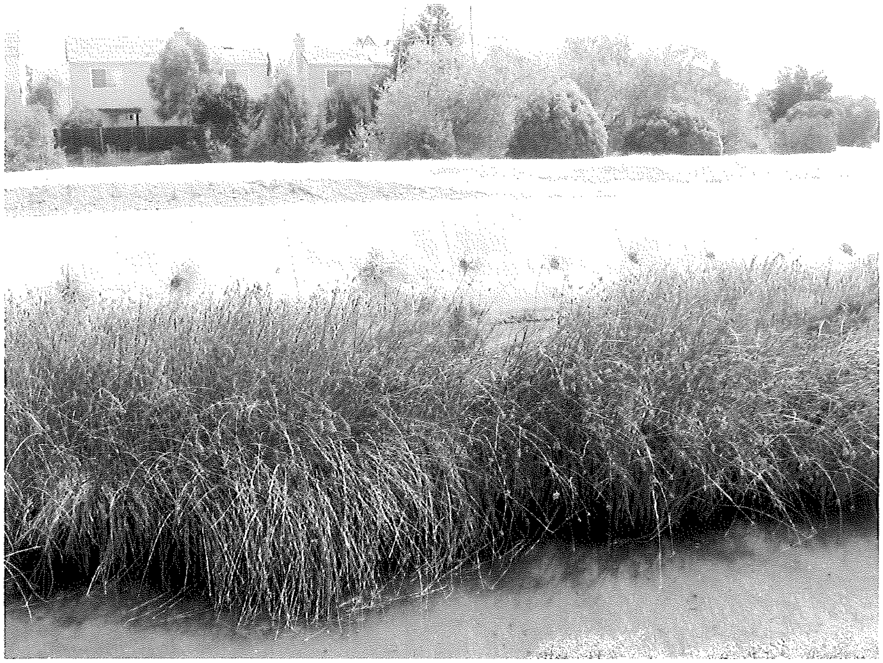
Photograph ID	Approximate Location (Project Station)	Description
114	00+80	View from opposite (west) bank (panorama photos 114 to 119)



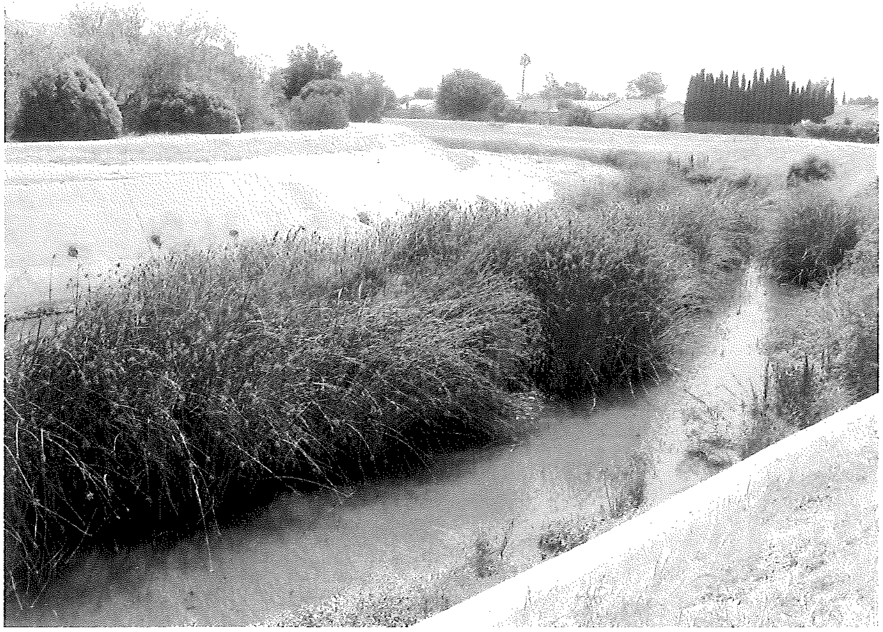
Photograph ID	Approximate Location (Project Station)	Description
115	00+90	View from opposite (west) bank (panorama photos 114 to 119)



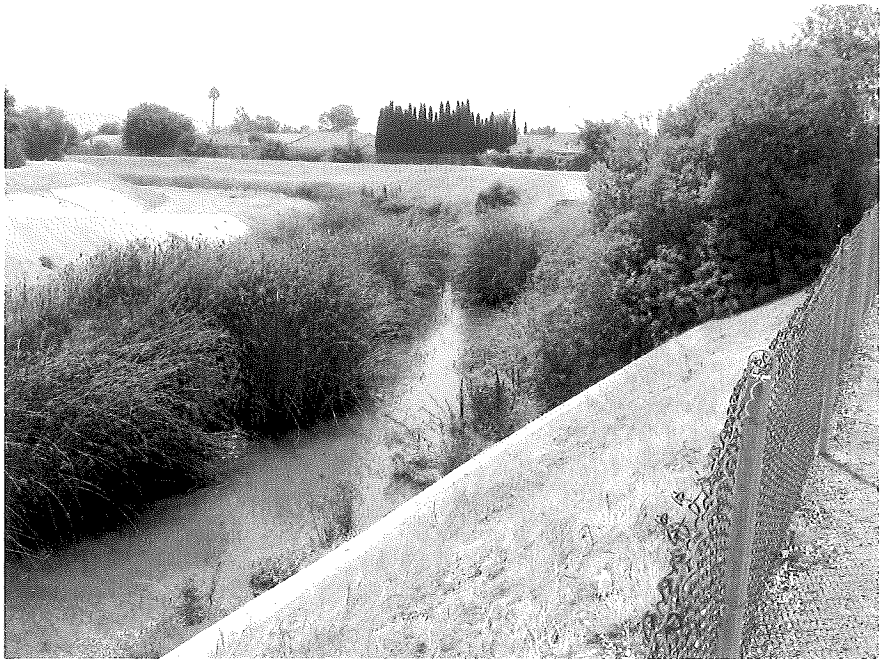
Photograph ID	Approximate Location (Project Station)	Description
116	00+90	View from opposite (west) bank (panorama photos 114 to 119)



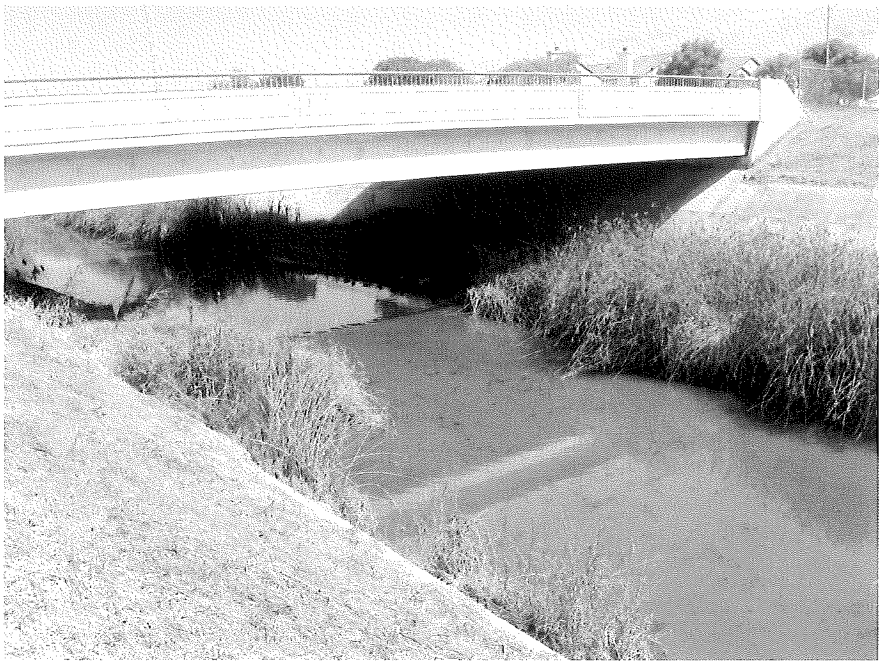
Photograph ID	Approximate Location (Project Station)	Description
117	00+90	View from opposite (west) bank (panorama photos 114 to 119)



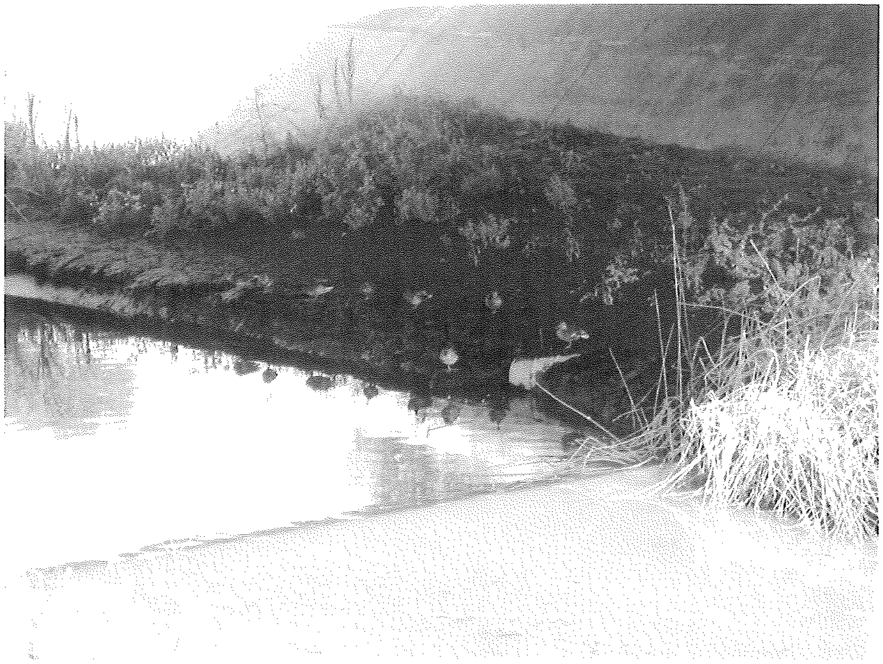
Photograph ID	Approximate Location (Project Station)	Description
118	00+90	View from opposite (west) bank (panorama photos 114 to 119)



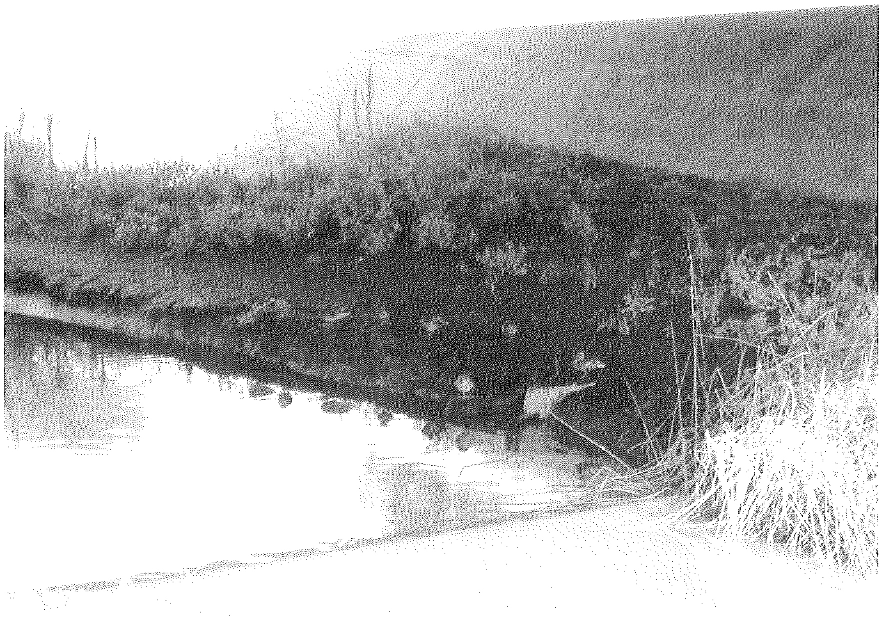
Photograph ID	Approximate Location (Project Station)	Description
119	00+90	View from opposite (west) bank (panorama photos 114 to 119)



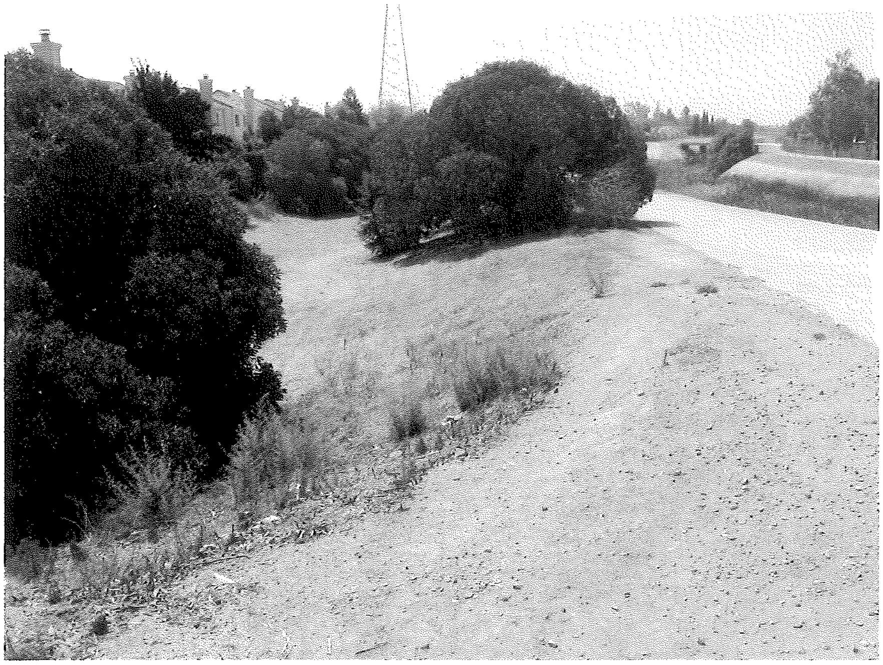
Photograph ID	Approximate Location (Project Station)	Description
120	41+30	Sediment buildup under bridge (from west bank)



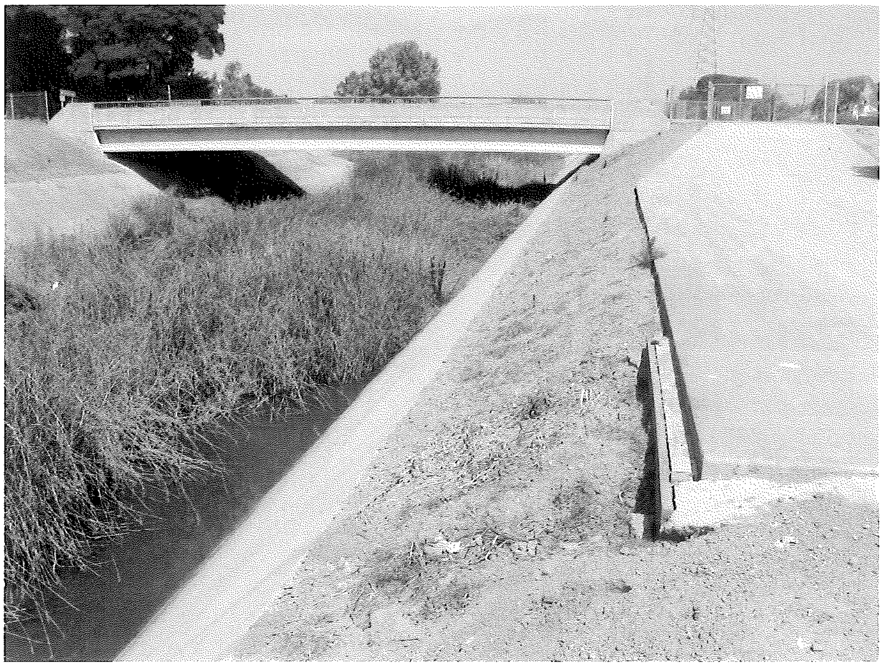
Photograph ID	Approximate Location (Project Station)	Description
121	41+30	Sediment buildup under bridge (from west bank)



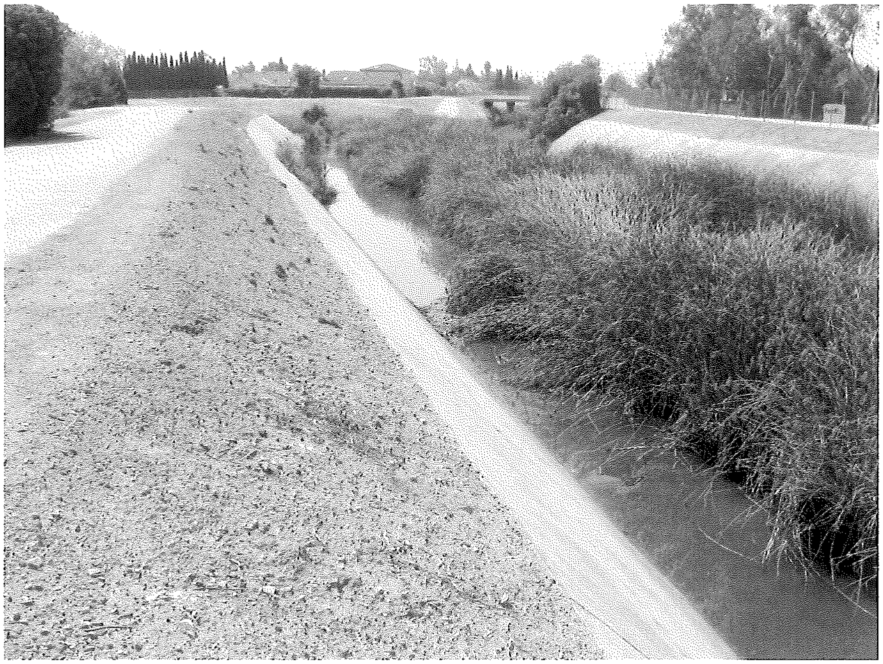
Photograph ID	Approximate Location (Project Station)	Description
122	41+30	Sediment buildup under bridge (from west bank)



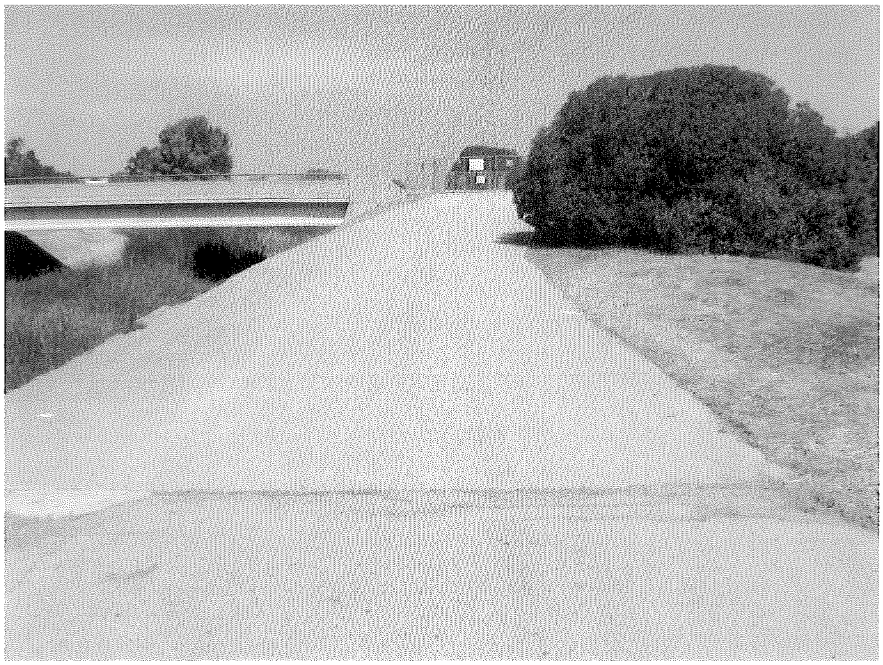
Photograph ID	Approximate Location (Project Station)	Description
123	40+80	South view of outboard levee slope (swale area)



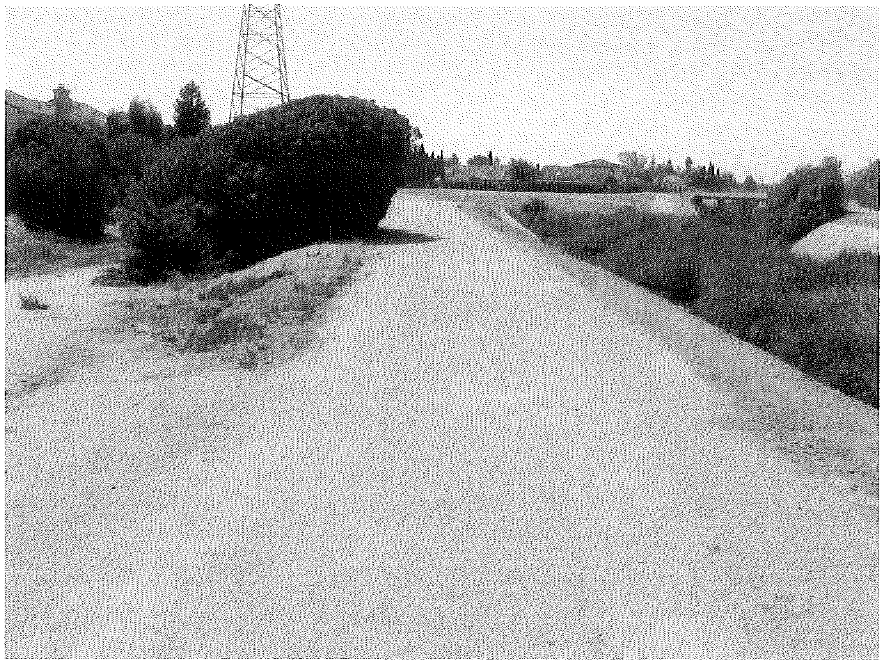
Photograph ID	Approximate Location (Project Station)	Description
124	42+40	North view of inboard levee slope



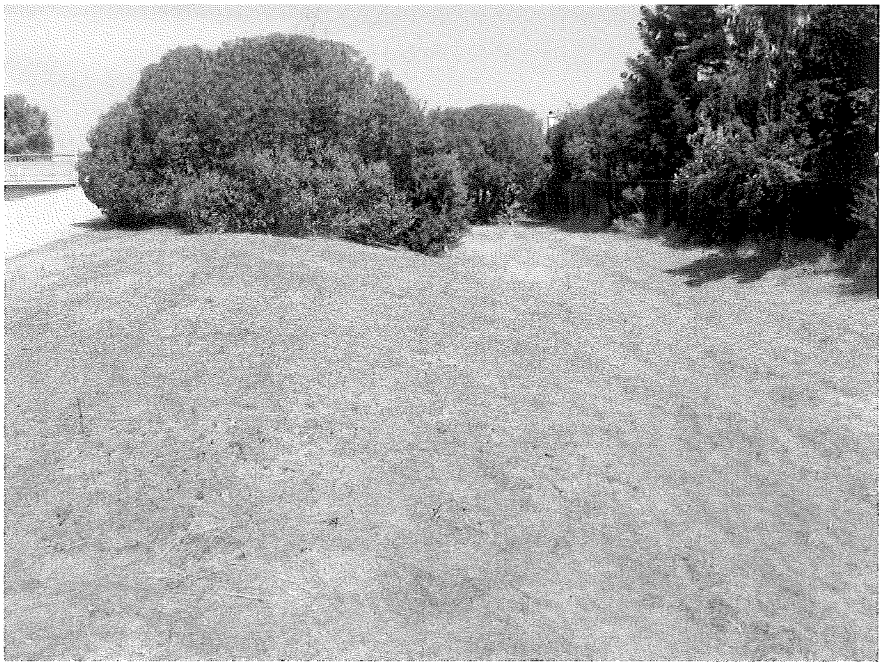
Photograph ID	Approximate Location (Project Station)	Description
125	42+40	South view of inboard levee slope



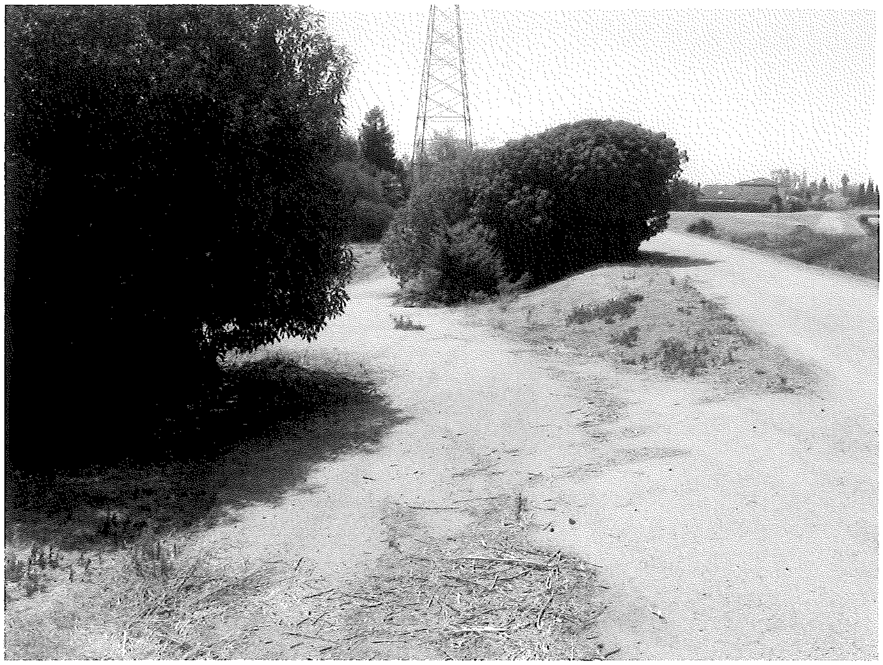
Photograph ID	Approximate Location (Project Station)	Description
126	42+40	North view of levee crest



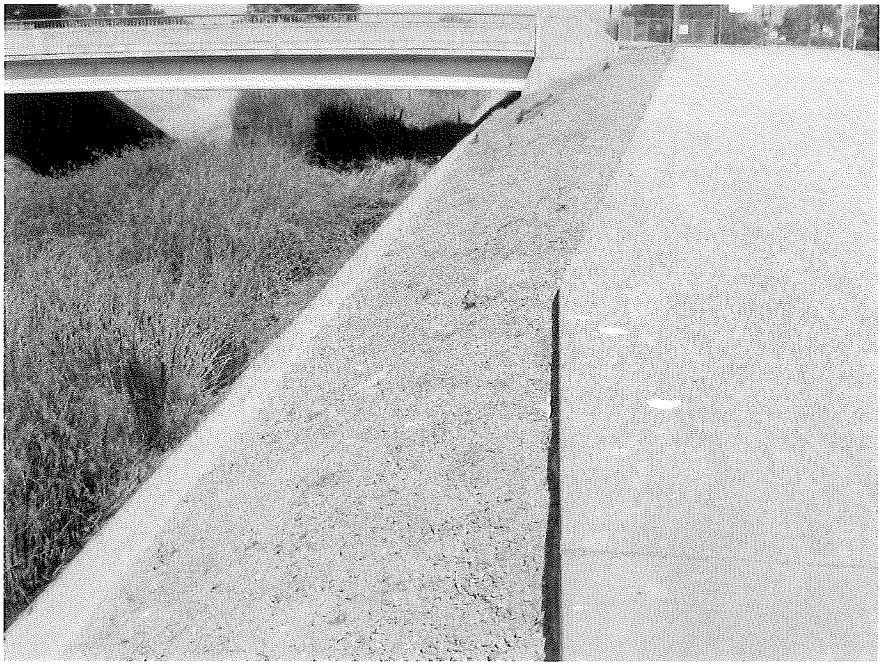
Photograph ID	Approximate Location (Project Station)	Description
127	42+40	South view of levee crest



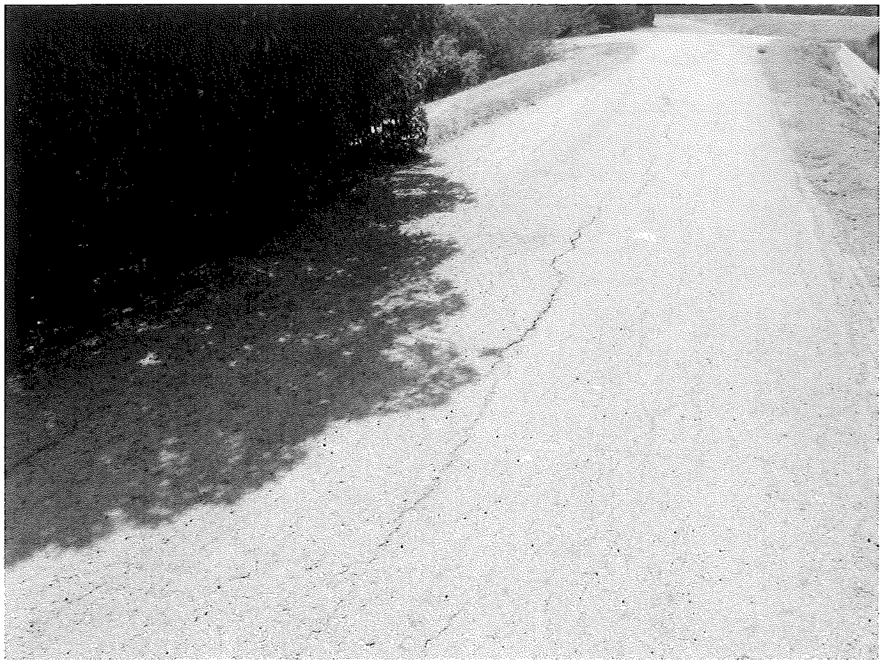
Photograph ID	Approximate Location (Project Station)	Description
128	42+40	North view of outboard levee slope



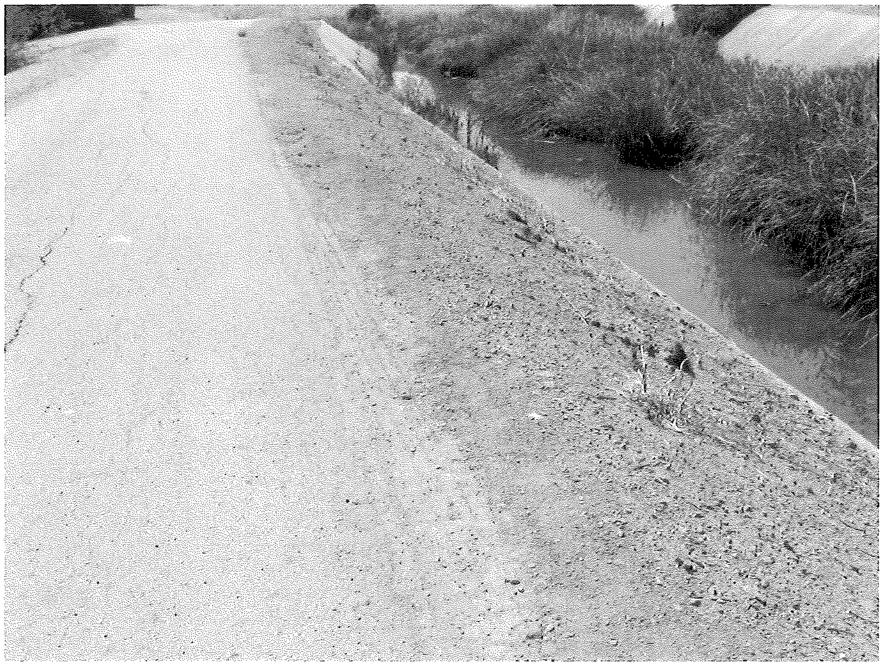
Photograph ID	Approximate Location (Project Station)	Description
129	42+40	South view of outboard levee slope



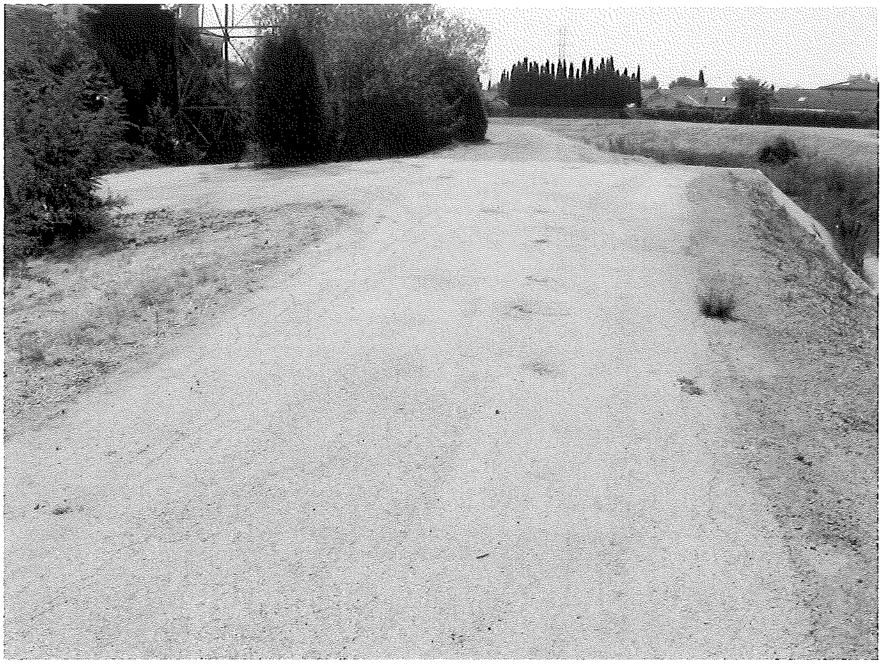
Photograph ID	Approximate Location (Project Station)	Description
130	41+90	North view of raveling, bare inboard levee slope



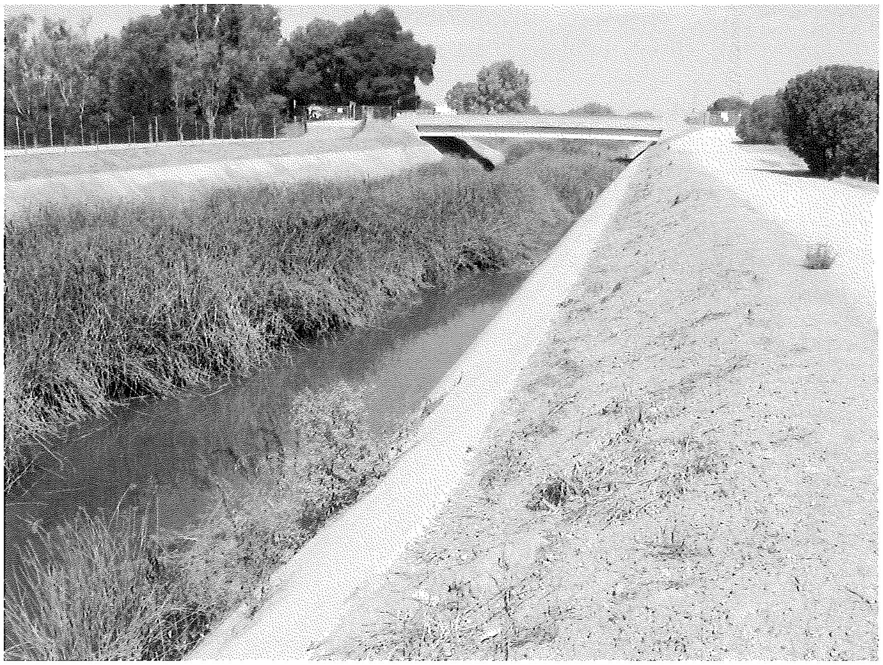
Photograph ID	Approximate Location (Project Station)	Description
131	43+00	Cracked pavement and bare raveling slope



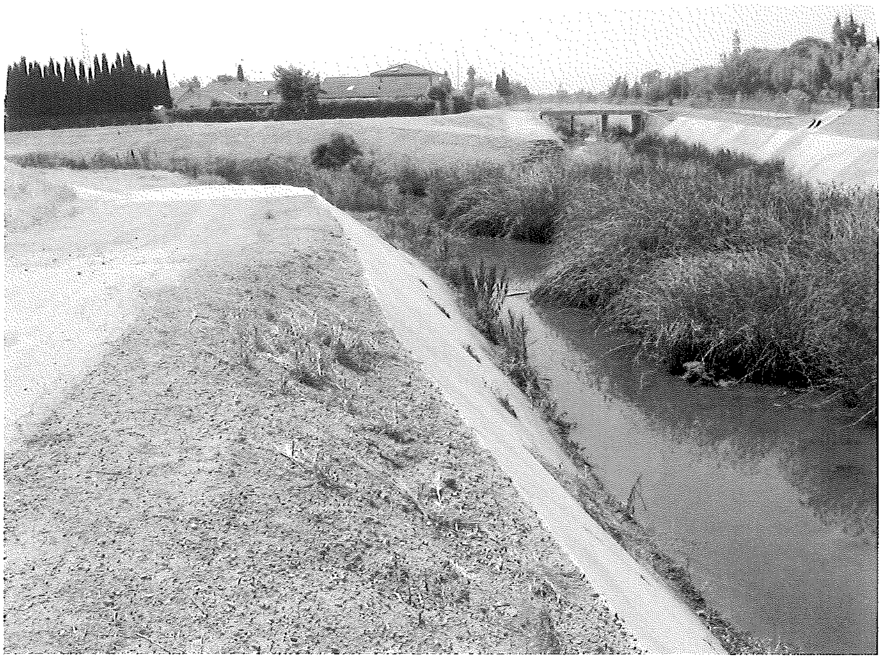
Photograph ID	Approximate Location (Project Station)	Description
132	0+00	Cracked/damaged pavement



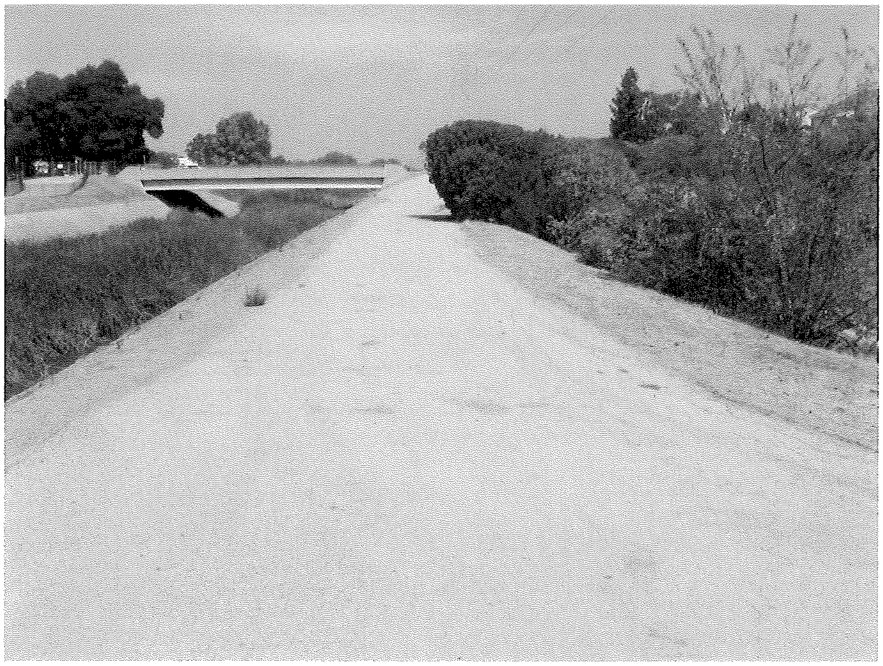
Photograph ID	Approximate Location (Project Station)	Description
133	0+50	South view of levee crest



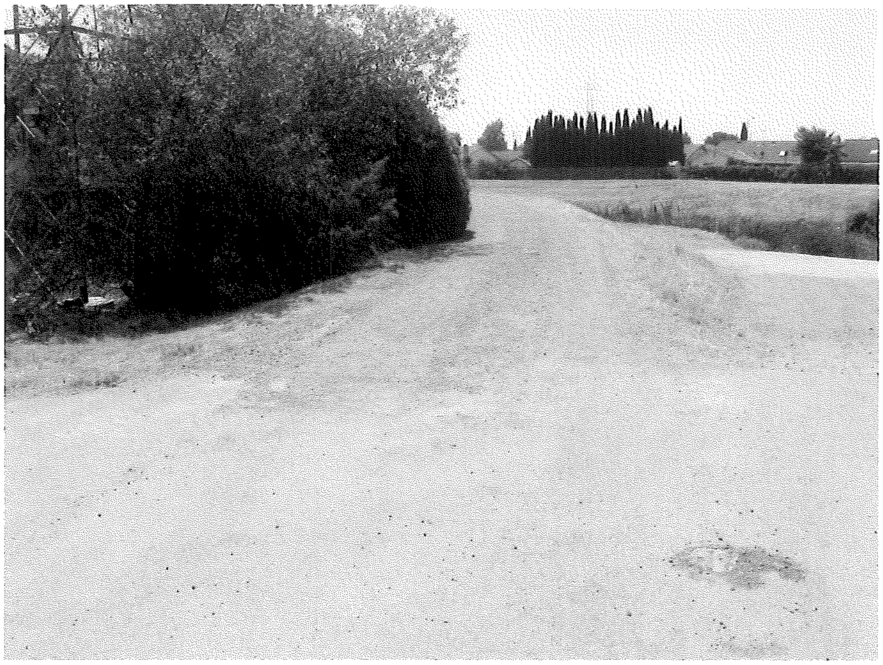
Photograph ID	Approximate Location (Project Station)	Description
134	0+50	North view of inboard levee slope



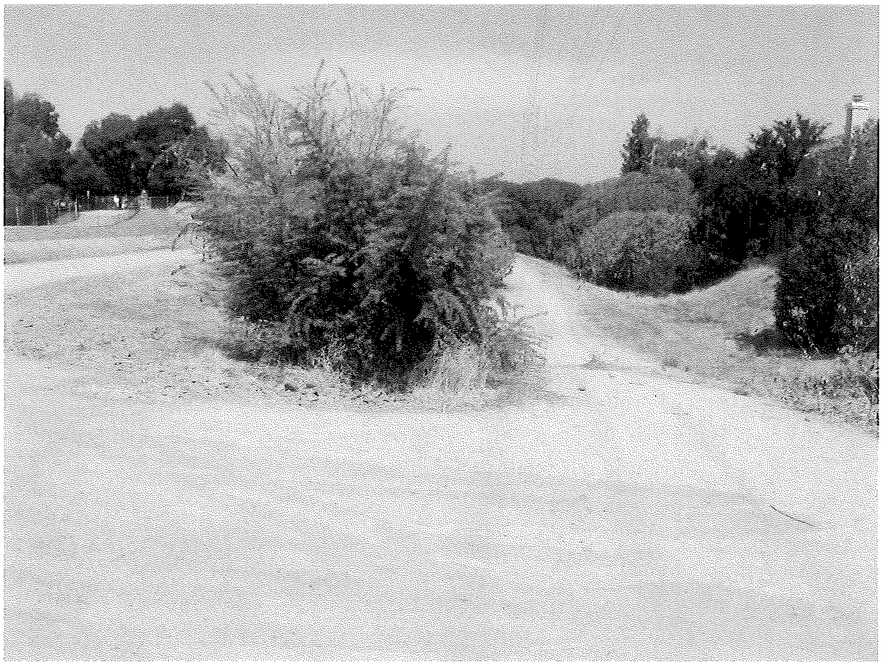
Photograph ID	Approximate Location (Project Station)	Description
135	0+50	South view of inboard levee slope



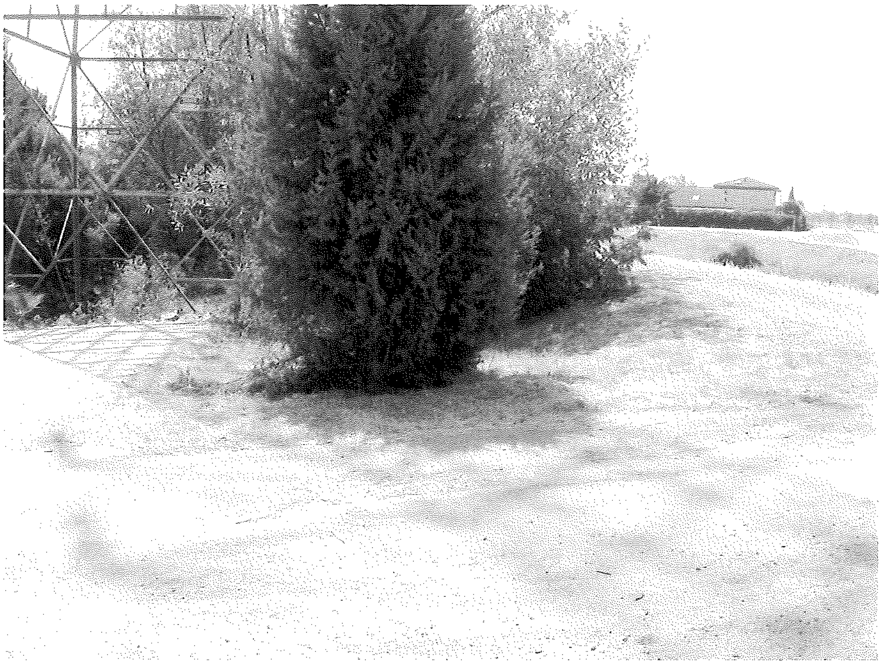
Photograph ID	Approximate Location (Project Station)	Description
136	0+50	North view of levee crest



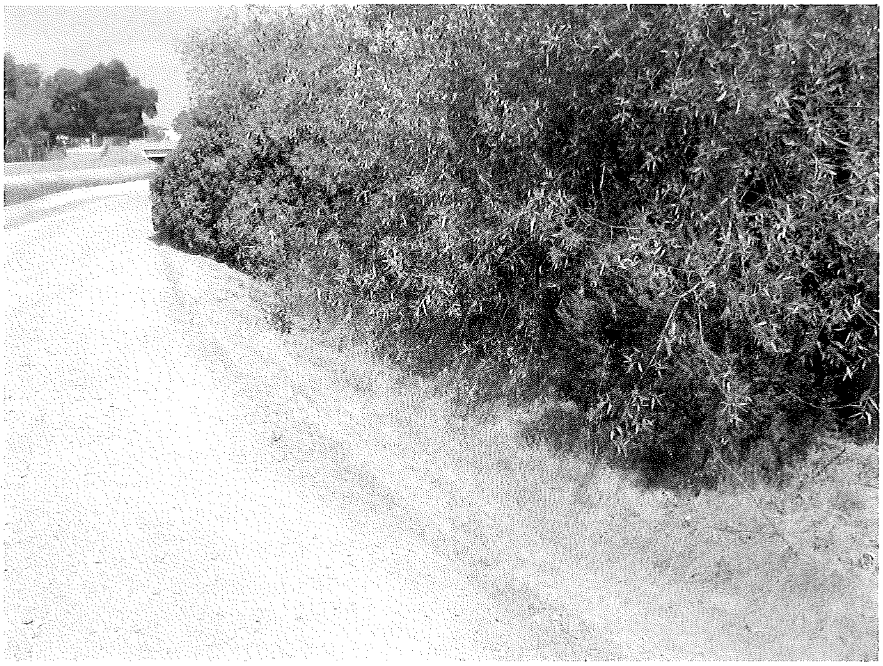
Photograph ID	Approximate Location (Project Station)	Description
137	0+50	South view of levee crest



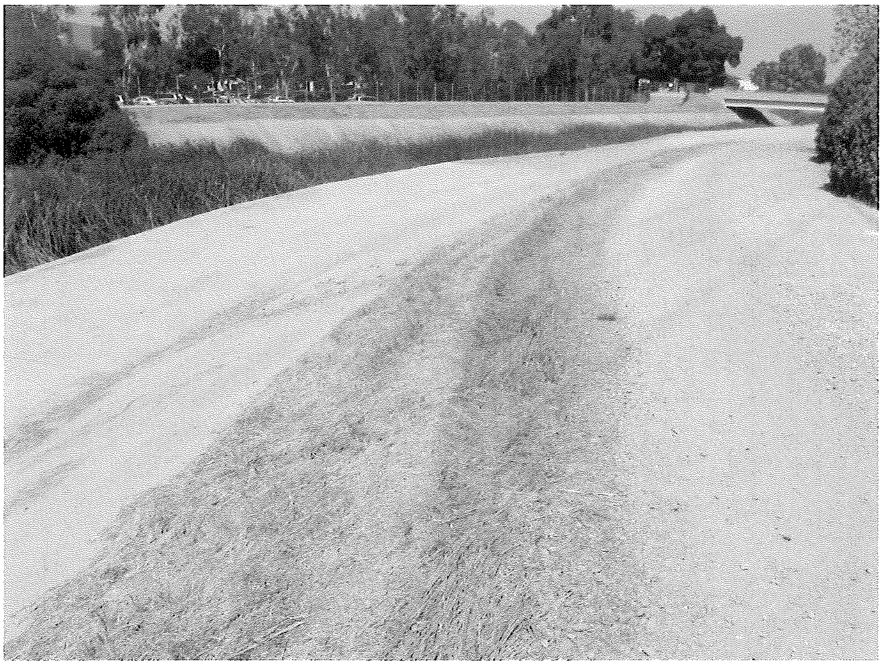
Photograph ID	Approximate Location (Project Station)	Description
138	0+50	North view of outboard levee slope



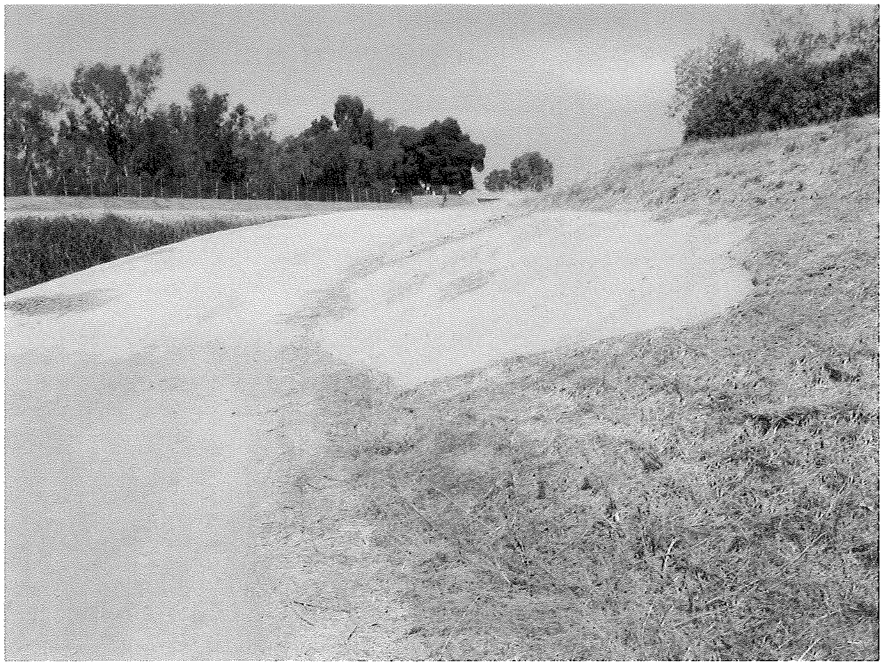
Photograph ID	Approximate Location (Project Station)	Description
139	0+50	South view of outboard levee slope



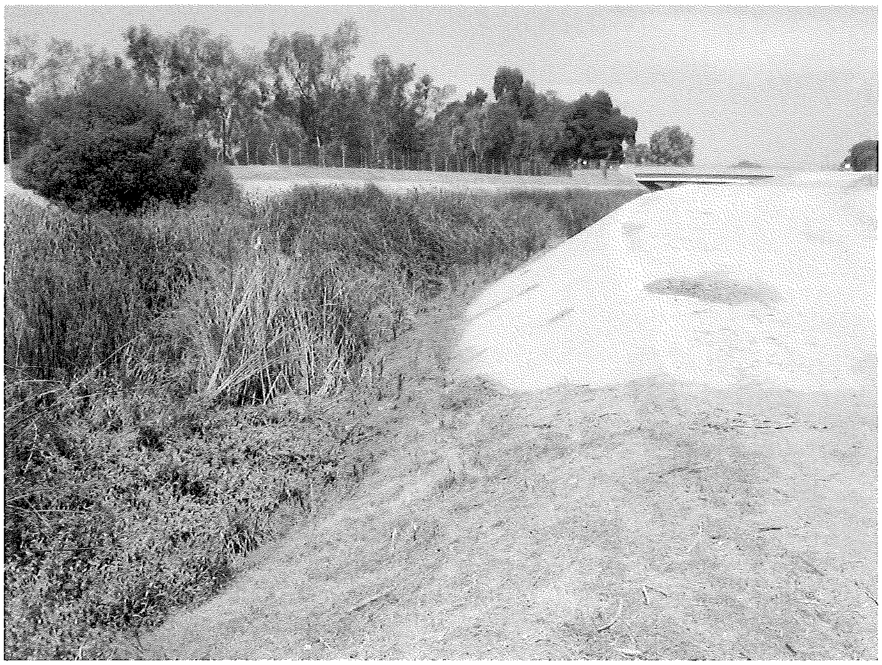
Photograph ID	Approximate Location (Project Station)	Description
140	2+30	North view of outboard levee slope



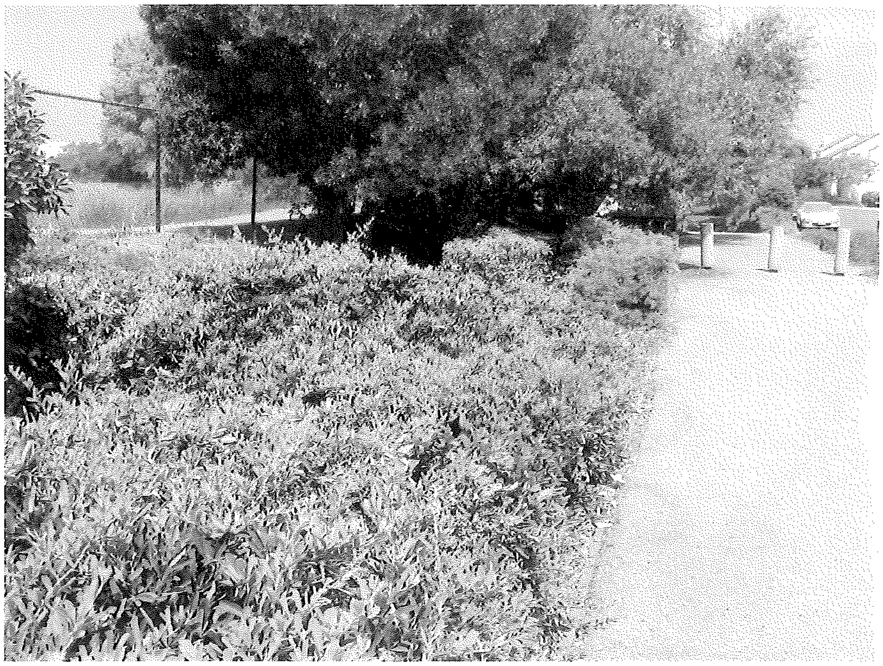
Photograph ID	Approximate Location (Project Station)	Description
141	2+30	North view of levee crest



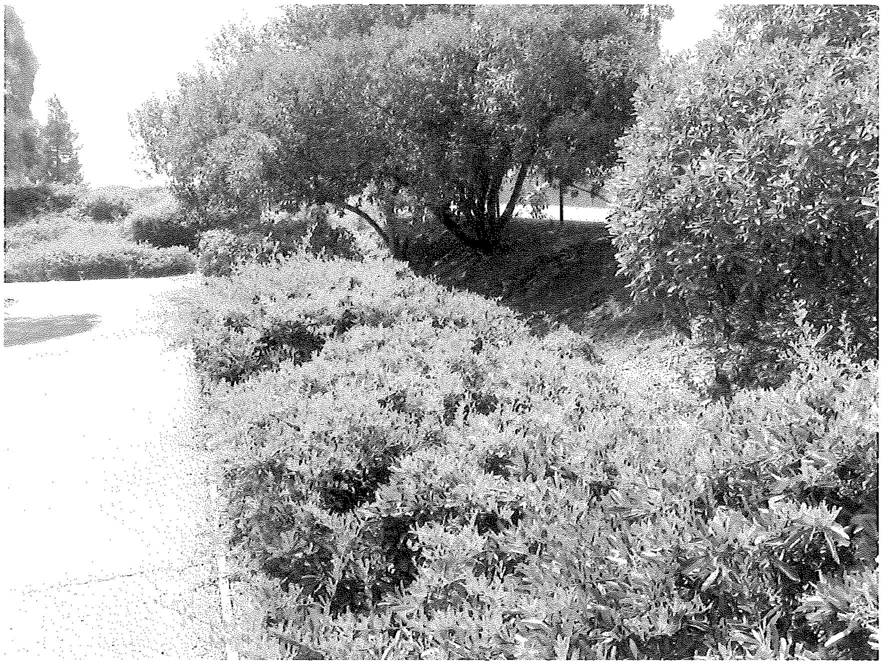
Photograph ID	Approximate Location (Project Station)	Description
142	2+30	North view of inboard levee slope



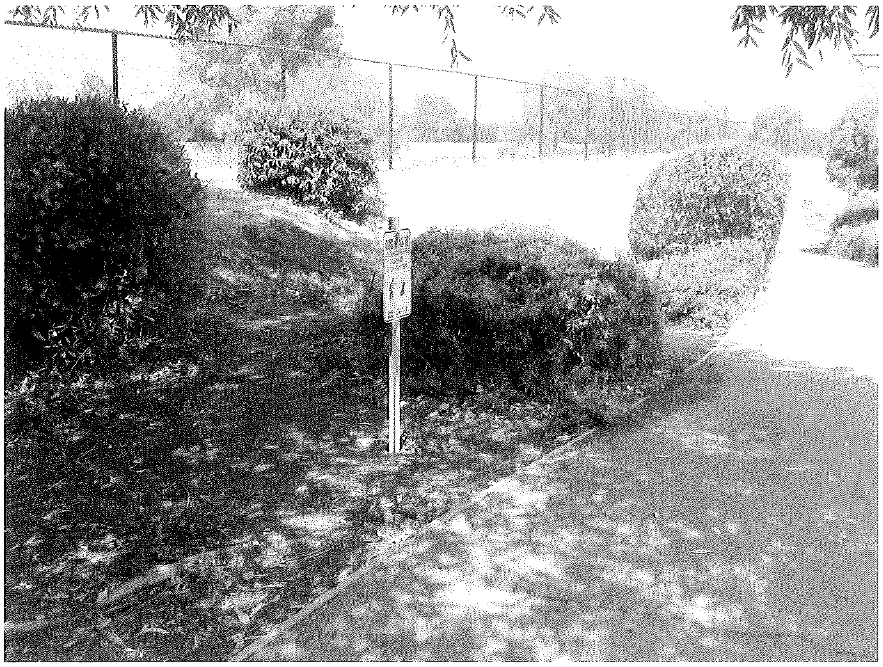
Photograph ID	Approximate Location (Project Station)	Description
143	2+30	North view of levee channel



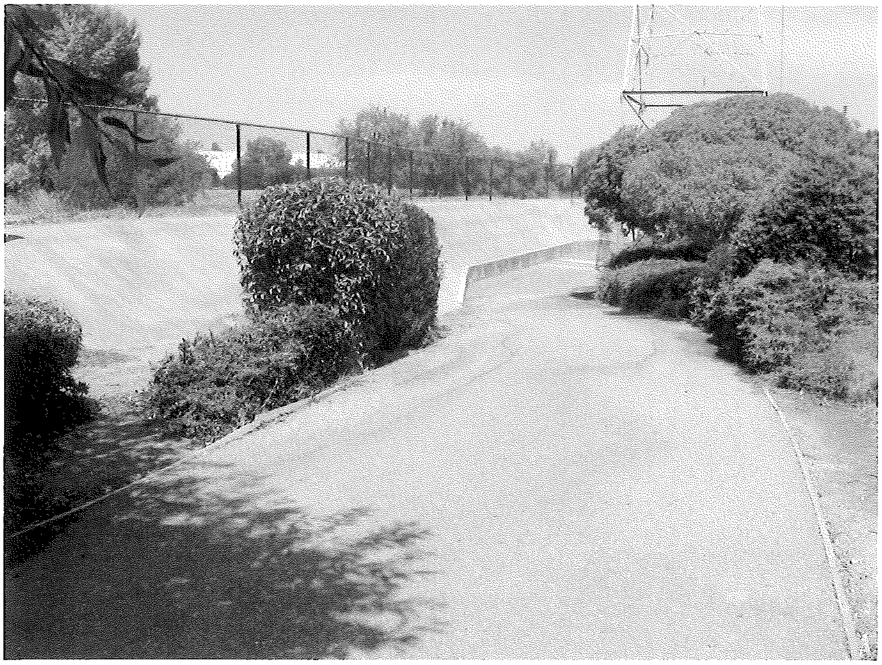
Photograph ID	Approximate Location (Project Station)	Description
144	39+20	North view of outboard levee slope



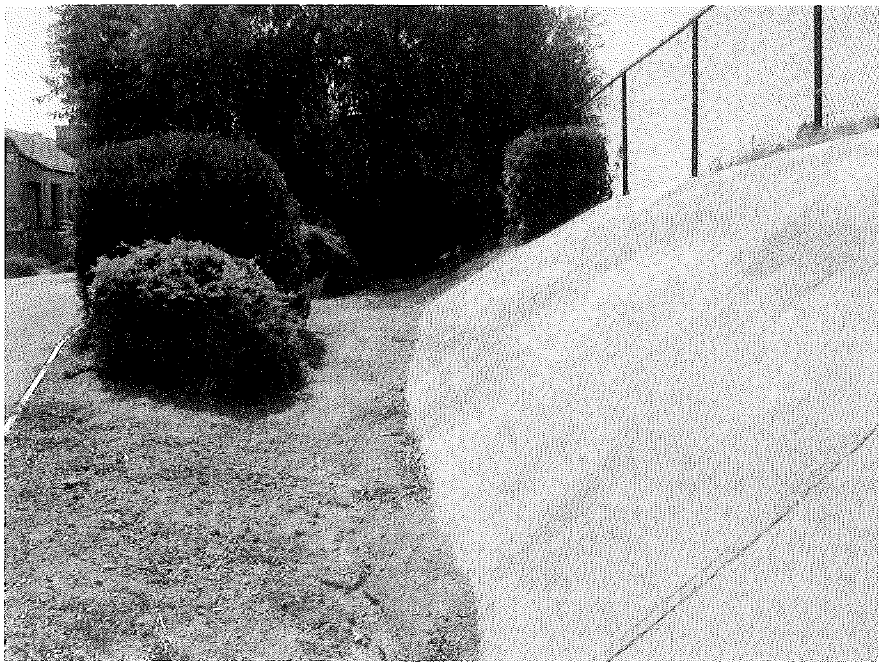
Photograph ID	Approximate Location (Project Station)	Description
145	39+20	South view of outboard levee slope



Photograph ID	Approximate Location (Project Station)	Description
146	39+00	North view of outboard levee slope



Photograph ID	Approximate Location (Project Station)	Description
147	38+20	North view of inboard levee concrete slope and retaining wall



Photograph ID	Approximate Location (Project Station)	Description
148	37+60	South view of outboard levee concrete slope

APPENDIX C

APPENDIX C

Field Exploration

ATTACHMENT C.1 Logs of Borings

ATTACHMENT C.2 Cone Penetration Test Logs

ATTACHMENT C.1

Logs of Borings

DATE: 10/10/2008	**LOG OF EXPLORATORY DRILL HOLE**	**Boring:** B-1

PROJECT NAME: SCVWD Lower Penitencia Ck Levee Recertification **PROJECT NUMBER:** 2227 E

DRILL RIG: CME 75, 140# auto hammer **LOGGED BY:** CSS

HOLE DIAMETER: 4" mud rotary **HOLE ELEVATION:** ---

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: **Initial:** Not established **Final:** Not established

DESCRIPTION OF EARTH MATERIALS (Lower maintenance road, approx. Sta. 30+50)	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
ALLUVIUM: **FAT CLAY:** Very dark gray (10YR 3/1), moist, very stiff	CH											
		1										
roughly 1' to 1.5' interval of dark grayish brown (10YR 4/2) clay		2	S D D	13	4.5 2.75							
		3										
stiff to very stiff		4	S D D	11	1.25 2.25			37		87		
		5										
grayish brown (10YR 5/2)		6	S D D	13	2.25- 3.0							
		7										
		8										
dark bluish gray (gley 2 3/10G) mottled with brown (7.5YR 4/4)		9	S D D	17	2.5- 3.25		55	28	37	99		
		10										
SILTY SAND: Dark greenish gray (gley2 3/10G), wet, loose; fine sand	SM	11										
		12										
		13										
NP = non-plastic * see Fig. B-3 for sieve+hydrometer results		14	S D D	6		42*		25	NP	101		
		15										
		16										
		17										
		18										
		19	S I I	8		21		26				
		20										

PACIFIC GEOTECHNICAL ENGINEERING **PAGE:** 1 of 2

DATE: 10/10/2008	**LOG OF EXPLORATORY DRILL HOLE**	**Boring:** B-1

PROJECT NAME: SCVWD Lower Penitencia Ck Levee Recertification **PROJECT NUMBER:** 2227 E

DRILL RIG: CME 75, 140# auto hammer **LOGGED BY:** CSS

HOLE DIAMETER: 4" mud rotary **HOLE ELEVATION:** ---

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: **Initial:** Not established **Final:** Not established

DESCRIPTION OF EARTH MATERIALS (Lower maintenance road, approx. Sta. 30+50)	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
SILTY SAND: continued	SM											
		21										
		22										
clayey and silty		23	S									
		24	I I	14		32		27				
POORLY GRADED SAND with GRAVEL and SILT: Dark Gray (5Y 4/1), wet, medium dense, fine to coarse sand; with fine gravel	SP-SM	25										
		26										
		27										
		28	S									
* see Fig. B-4 for sieve analysis results		29	I I	19		8*		18				
		30										
CLAYEY SAND: Very dark gray (3Y 3/1), wet, medium dense; fine to coarse sand; with fine gravel	SC	31										
		32										
		33	S									
		34	I I	21		30		22				
LEAN CLAY: Gray (10YR 5/1) mottled with brown (7.5YR 4/4), wet, medium dense	CL	35										
		36	S									
		37	I I	16		93	38	30	20			
BOTTOM OF HOLE = 37.5 Feet Groundwater depth cannot be determined due to mud rotary drilling method		38										
		39										
		40										

DATE: 10/10/2008	**LOG OF EXPLORATORY DRILL HOLE**	**Boring:** B-2

PROJECT NAME:	SCVWD Lower Penitencia Ck Levee Recertification	**PROJECT NUMBER:**	2227 E
DRILL RIG:	CME 75, 140# auto hammer	**LOGGED BY:**	CSS
HOLE DIAMETER:	4" mud rotary	**HOLE ELEVATION:**	---

SAMPLER:	D = 3" OD, 2½" ID Split-spoon X = 2½" OD, 2" ID Split-spoon I = Standard Penetrometer (2" OD SPT) S = Slough in sample	**GROUND WATER DEPTH:**	**Initial:** **Final:**	Not established Not established

DESCRIPTION OF EARTH MATERIALS (On top of levee , approx. Sta. 42+00)	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
FILL: SANDY FAT CLAY: Very dark gray (10YR 3/1) to very dark brown (10YR 3/2), moist, very stiff; with mostly fine to medium sand * see Fig. B-5 for sieve analysis results	CH											
		1										
		2	S D D	26		70	50*	22	27	96		
		3										
		4	S D D	34								
ALLUVIUM: LEAN CLAY with SAND: Black (10YR 2/1), moist, hard; with mostly fine to medium sand	CL	5										
		6	S D D	44	4.5+	77	48	28 23	28	103	15	(TXUU) 10542
		7										
		8										
		9	S D D	54	4.5+							
		10										
FAT CLAY: Black (10YR 2/1), moist, very stiff	CH	11										
		12										
		13										
		14	S D D	14	2.75 3.5			30		95		
		15										
SILT: Grayish brown (10YR 5/2) mottled with brown (7.5Y 4/4), moist to wet, medium dense	ML	16										
		17										
		18										
		19	S D D	14	1.1			26		101		
		20										

PACIFIC GEOTECHNICAL ENGINEERING **PAGE:** 1 of 2

DATE: 10/10/2008

LOG OF EXPLORATORY DRILL HOLE

Boring: B-2

PROJECT NAME: SCVWD Lower Penitencia Ck Levee Recertification

PROJECT NUMBER: 2227 E

DRILL RIG: CME 75, 140# auto hammer

LOGGED BY: CSS

HOLE DIAMETER: 4" mud rotary

HOLE ELEVATION: ---

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH:
Initial: Not established
Final: Not established

DESCRIPTION OF EARTH MATERIALS (On top of levee , approx. Sta. 42+00)	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
SILT: continued	ML											
		21										
NP = non-plastic * see Fig. B-6 for sieve analysis results see Fig. B-12 for consolidation test results		22	Shelby Tube									
		23		400 psi		90*		28 28	NP	96 95		TXCU (see Fig. B-11)
		24										
		25										
		26										
LEAN CLAY: Brown (10YR 4/3), moist to wet, very stiff	CL	27										
		28	S									
		29	D D	30	4.0 2.5	98		25		103		
BOTTOM OF HOLE = 29.5 Feet Groundwater depth cannot be determined due to mud rotary drilling method		30										
		31										
		32										
		33										
		34										
		35										
		36										
		37										
		38										
		39										
		40										

PACIFIC GEOTECHNICAL ENGINEERING

PAGE: 2 of 2

DATE: 10/10/2008	**LOG OF EXPLORATORY DRILL HOLE**	**Boring:** B-3

PROJECT NAME: SCVWD Lower Penitencia Ck Levee Recertification	**PROJECT NUMBER:** 2227 E
DRILL RIG: CME 75, 140# auto hammer	**LOGGED BY:** CSS
HOLE DIAMETER: 4" mud rotary	**HOLE ELEVATION:** ---
SAMPLER: D = 3" OD, 2½" ID Split-spoon X = 2½" OD, 2" ID Split-spoon I = Standard Penetrometer (2" OD SPT) S = Slough in sample	**GROUND WATER DEPTH:** **Initial:** Not established **Final:** Not established

DESCRIPTION OF EARTH MATERIALS (Lower maintenance road, approx. Sta. 21+40)	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
ALLUVIUM: **FAT CLAY:** Very dark gray (10YR 3/1) to very dark brown (10YR 3/2), moist, very stiff to hard	CH	1	S									
dark grayish brown (10YR 4/2)		2	D D	16	3.5- 4.5							
		3	S									
black (10YR 2/1), moist, very stiff		4	D D	15	2.75- 3.25			33		90		
		5	S									
very dark gray (10YR 3/1)		6	D D	9	3.0- 3.5		71	44	48	77	6	(TXUU) 2553
		7										
		8	S									
Dark gray (10YR 4/1) to gray (5Y 5/1) mottled with dark yellowish brown (10YR 4/4)		9	D D	16	1.25 3.5			24		103		
		10										
		11										
SILTY SAND: Very dark gray (10YR 3/1), wet, medium dense; mostly fine sand	SP	12										
		13	S									
		14	D D	13		30*		22		106		
* see Fig. B-7 for sieve analysis results		15										
		16										
		17										
		18	S									
		19	I I	15				27				
		20										

PACIFIC GEOTECHNICAL ENGINEERING | **PAGE:** 1 of 2

DATE: 10/10/2008	**LOG OF EXPLORATORY DRILL HOLE**	**Boring:** B-3

PROJECT NAME: SCVWD Lower Penitencia Ck Levee Recertification	**PROJECT NUMBER:** 2227 E
DRILL RIG: CME 75, 140# auto hammer	**LOGGED BY:** CSS
HOLE DIAMETER: 4" mud rotary	**HOLE ELEVATION:** ---

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: **Initial:** Not established **Final:** Not established

DESCRIPTION OF EARTH MATERIALS (Lower maintenance road, approx. Sta. 21+40)	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
SILTY SAND: continued	SM											
		21										
		22										
		23	S									
		24	I I	19		14		28				
		25										
POORLY GRADED SAND with CLAY: Dark gray (5Y 4/1), wet, medium dense; fine to coarse sand * see Fig. B-8 for sieve analysis results	SP-SC	26										
		27										
		28	S									
		29	I I	25		9*		22				
		30										
SILTY SAND with GRAVEL: bluish black (gley 2 2.5/10B), wet, medium dense, fine to coarse sand; with fine gravel ** see Fig. B-9 for sieve analysis results	SM	31										
		32										
		33	S									
SILTY CLAY: Bluish black, wet, very stiff	CL-ML	34	I I	20		15**	24	14	6			
		35										
BOTTOM OF HOLE = 34.5 Feet Groundwater depth cannot be determined due to mud rotary drilling method		36										
		37										
		38										
		39										
		40										

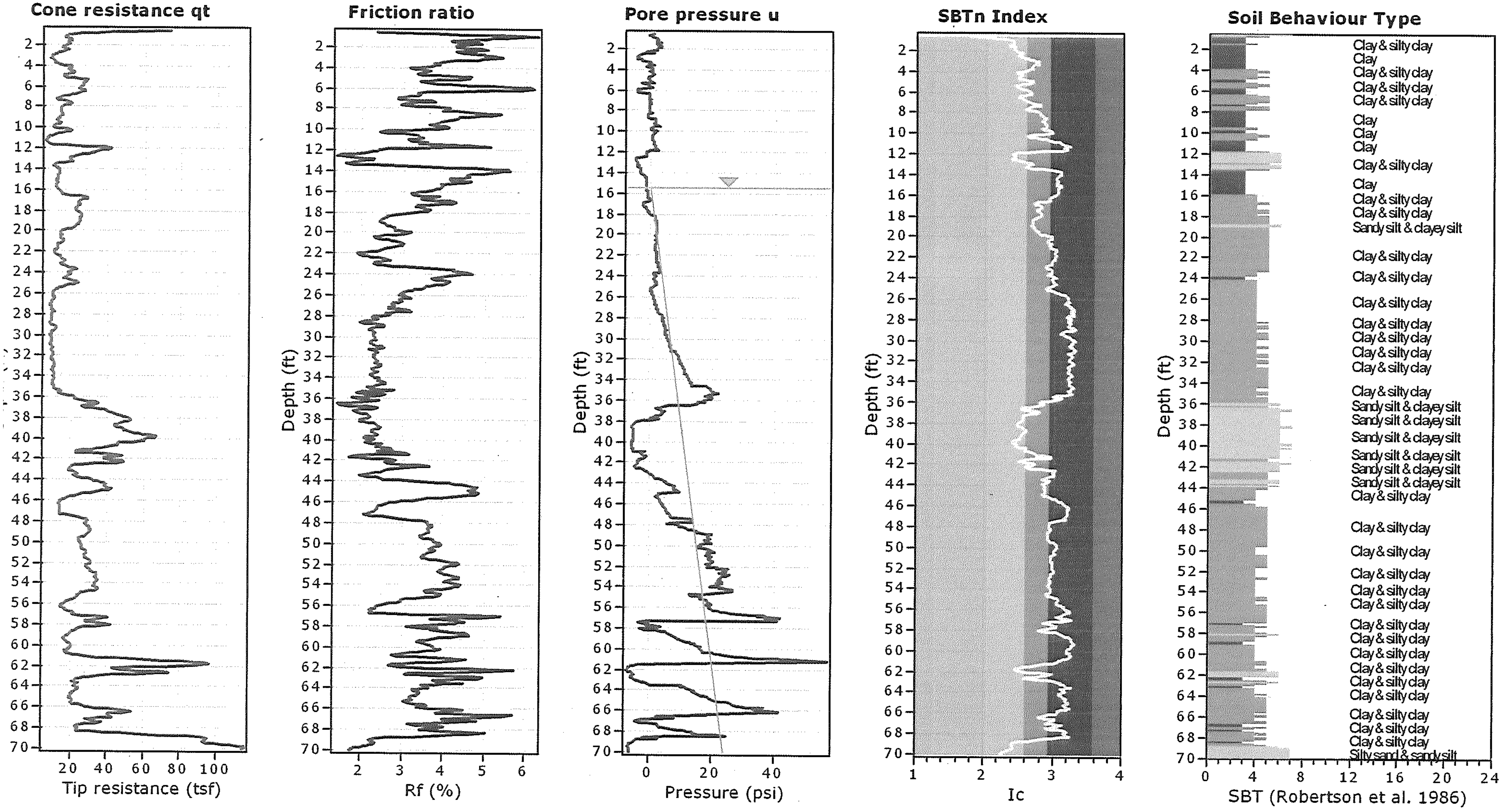
ATTACHMENT C.2

Cone Penetration Test Logs

AMEC Geomatrix
2101 Webster St 12th Floor
Oakland CA 94612

roject: Penitencia Ck Levee Milpitas

CPT: CPT-01 - Total depth: 70.01 (ft)



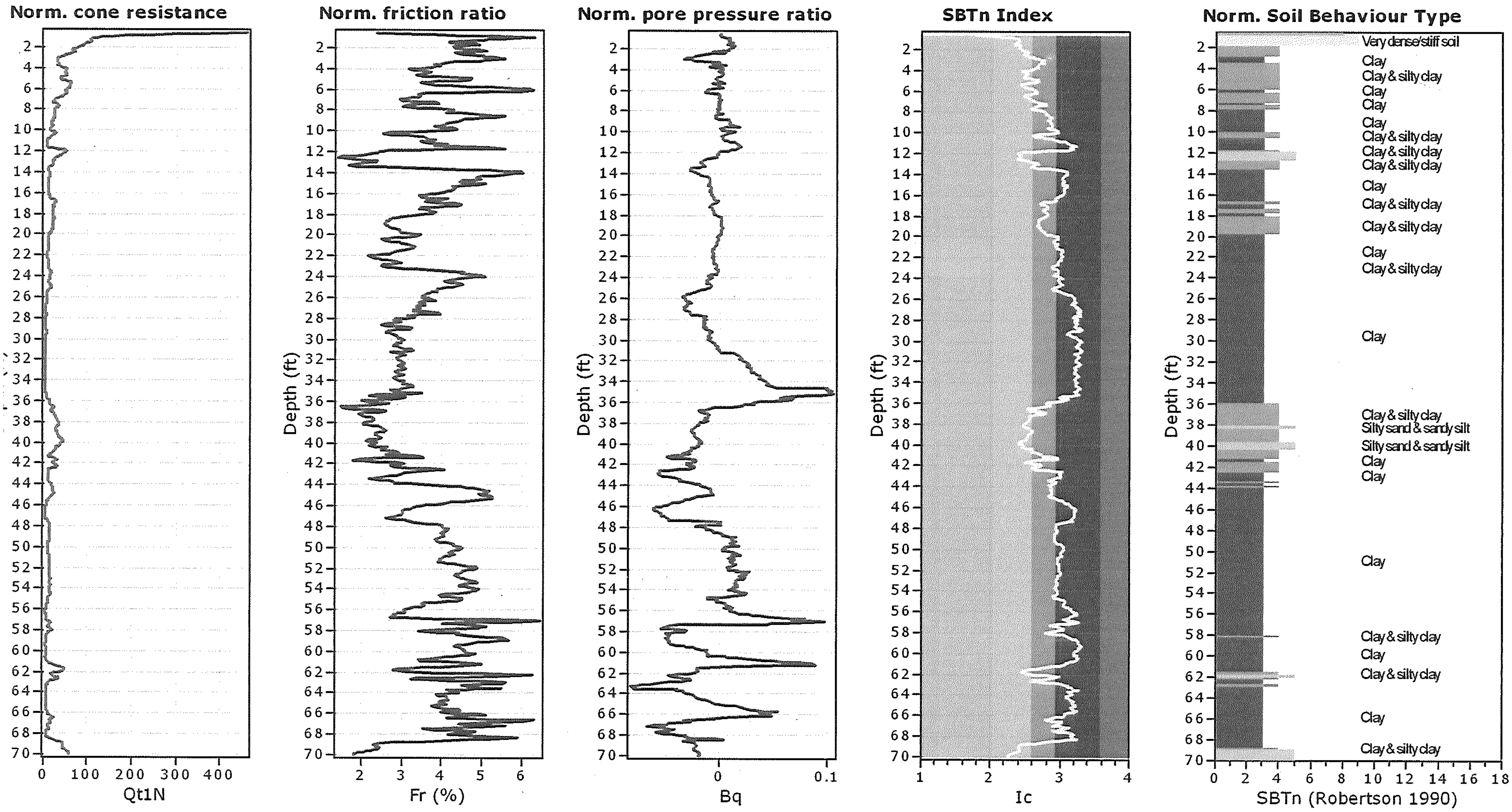
SBT legend

- ▨ 1. Sensitive fine grained
- ▨ 2. Organic material
- ▨ 3. Clay
- ▨ 4. Clay & silty clay
- ▨ 5. Clay & silty clay
- ▨ 6. Sandy silt & clayey silt
- ▨ 7. Silty sand & sandy silt
- ▨ 8. Sand & silty sand
- ▨ 9. Sand
- ▨ 10. Sand
- □ 11. Very dense/stiff soil
- ▨ 12. Very dense/stiff soil

PeT-IT v.1.6.0.68 - CPTU data presentation & interpretation software - Report created on: 6/3/2009, 1:53:36 PM
oject file: I:\Project\13000\13815.003\12000 CALC EVAL ANALYSES\Cliq\Penitencia Ck Levee Milpitas cpt.cpt

AMEC Geomatrix
2101 Webster St 12th Floor
Oakland CA 94612

roject: Penitencia Ck Levee Milpitas **CPT: CPT-01 - Total depth: 70.01 (ft)**



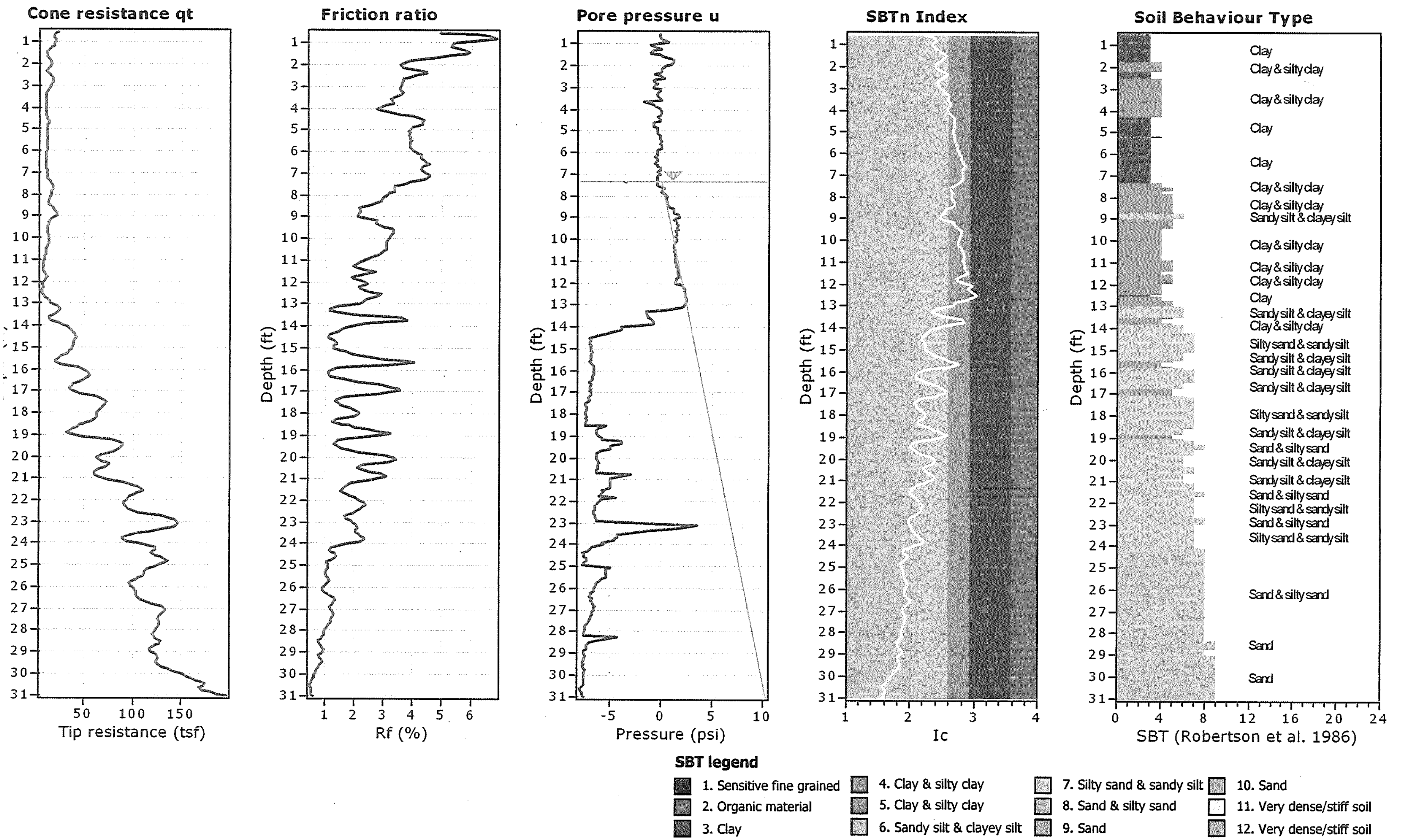
SBTn legend

▨ 1. Sensitive fine grained	▨ 4. Clayey silt to silty clay	▨ 7. Gravely sand to sand
▨ 2. Organic material	▨ 5. Silty sand to sandy silt	▨ 8. Very stiff sand to clayey sand
▨ 3. Clay to silty clay	▨ 6. Clean sand to silty sand	☐ 9. Very stiff fine grained

AMEC Geomatrix
2101 Webster St 12th Floor
Oakland CA 94612

roject: Penitencia Ck Levee Milpitas

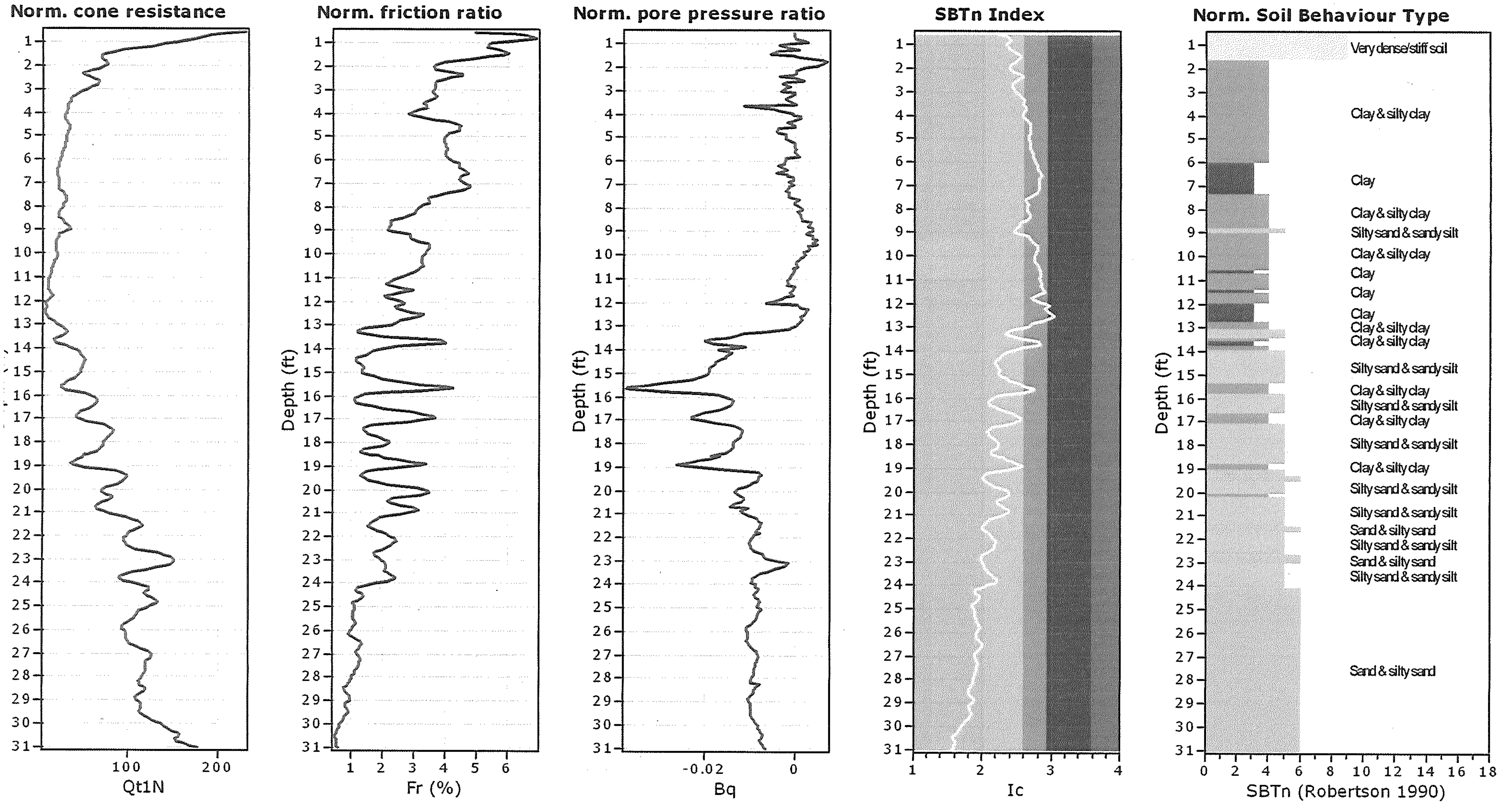
CPT: CPT-02 - Total depth: 31.02 (ft)



AMEC Geomatrix
2101 Webster St 12th Floor
Oakland CA 94612

roject: Penitencia Ck Levee Milpitas

CPT: CPT-02 - Total depth: 31.02 (ft)



SBTn legend

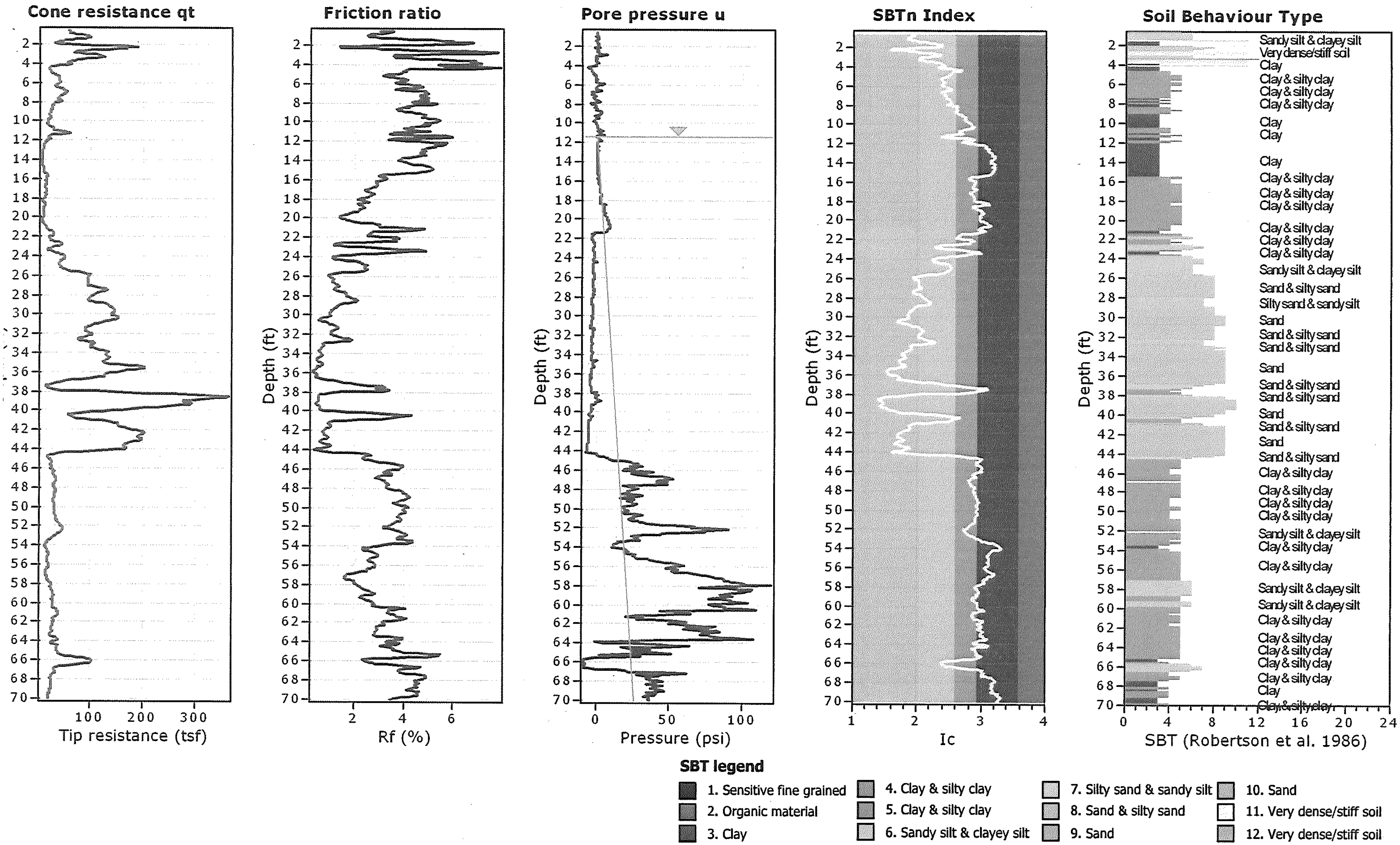
- ■ 1. Sensitive fine grained
- ■ 2. Organic material
- ■ 3. Clay to silty clay
- ■ 4. Clayey silt to silty clay
- ■ 5. Silty sand to sandy silt
- ■ 6. Clean sand to silty sand
- ■ 7. Gravely sand to sand
- ■ 8. Very stiff sand to clayey sand
- □ 9. Very stiff fine grained

PeT-IT v.1.6.0.68 - CPTU data presentation & interpretation software - Report created on: 6/3/2009, 1:53:37 PM
oject file: I:\Project\13000\13815.003\12000 CALC EVAL ANALYSES\Cliq\Penitencia Ck Levee Milpitas cpt.cpt

AMEC Geomatrix
2101 Webster St 12th Floor
Oakland CA 94612

roject: Penitencia Ck Levee Milpitas

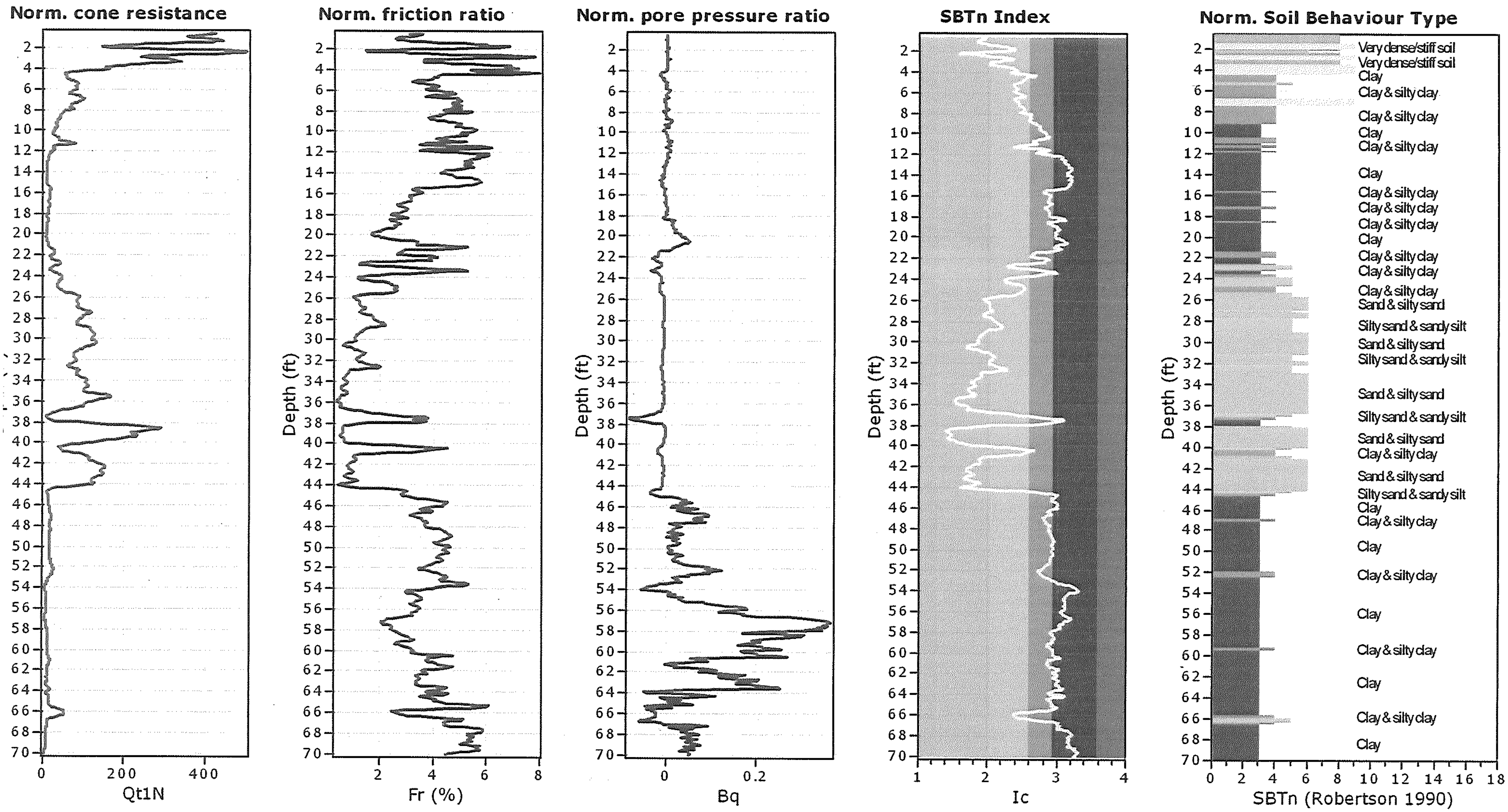
CPT: CPT-03 - Total depth: 70.06 (ft)



AMEC Geomatrix
2101 Webster St 12th Floor
Oakland CA 94612

roject: **Penitencia Ck Levee Milpitas**

CPT: CPT-03 - Total depth: 70.06 (ft)



SBTn legend

- ■ 1. Sensitive fine grained
- ■ 2. Organic material
- ■ 3. Clay to silty clay
- ■ 4. Clayey silt to silty clay
- ■ 5. Silty sand to sandy silt
- ■ 6. Clean sand to silty sand
- ■ 7. Gravely sand to sand
- ■ 8. Very stiff sand to clayey sand
- □ 9. Very stiff fine grained

PeT-IT v.1.6.0.68 - CPTU data presentation & interpretation software - Report created on: 6/3/2009, 1:53:38 PM
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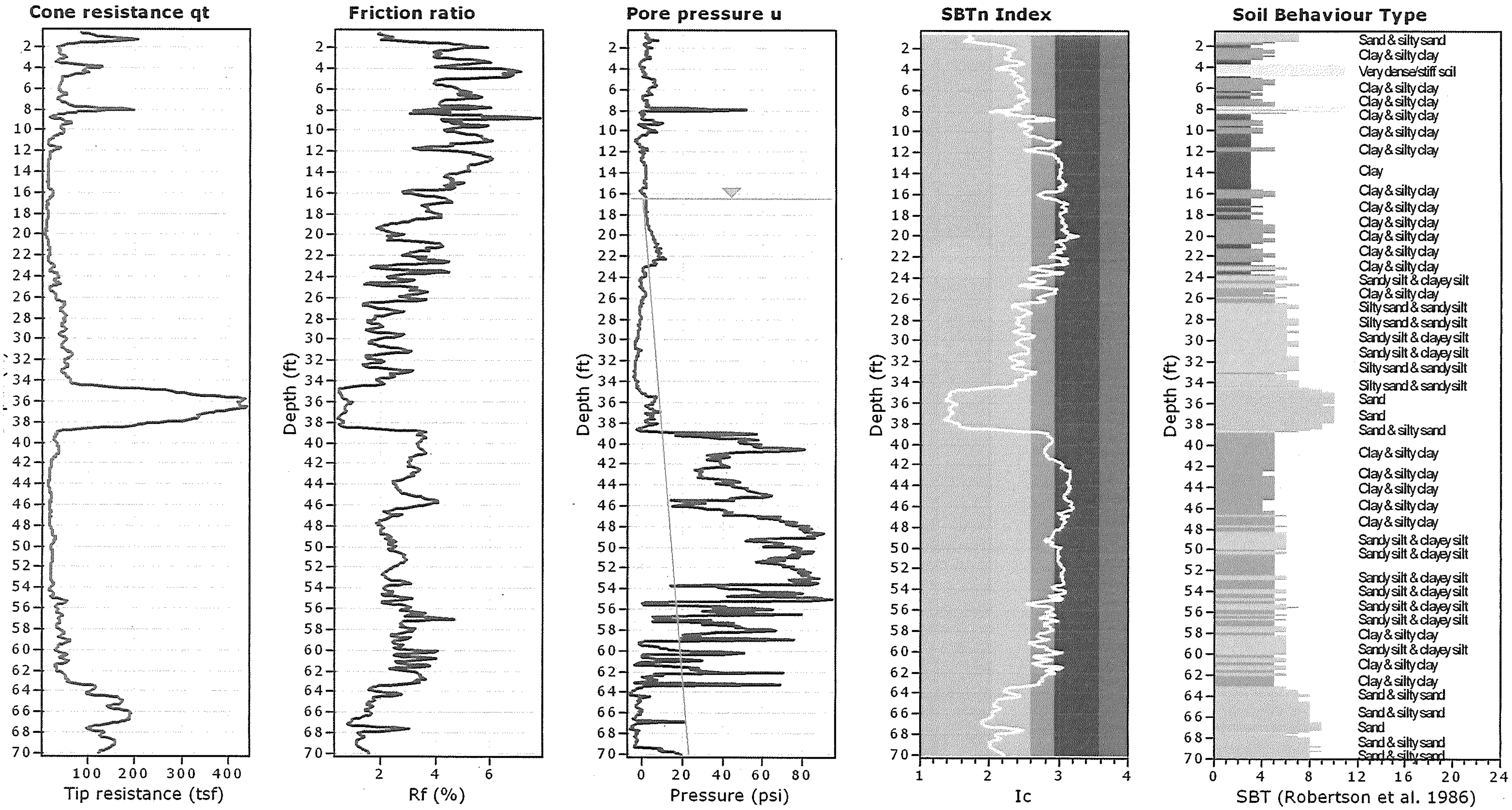
AMEC Geomatrix
2101 Webster St 12th Floor
Oakland CA 94612

roject: Penitencia Ck Levee Milpitas **CPT: CPT-04 - Total depth: 70.03 (ft)**



SBT legend

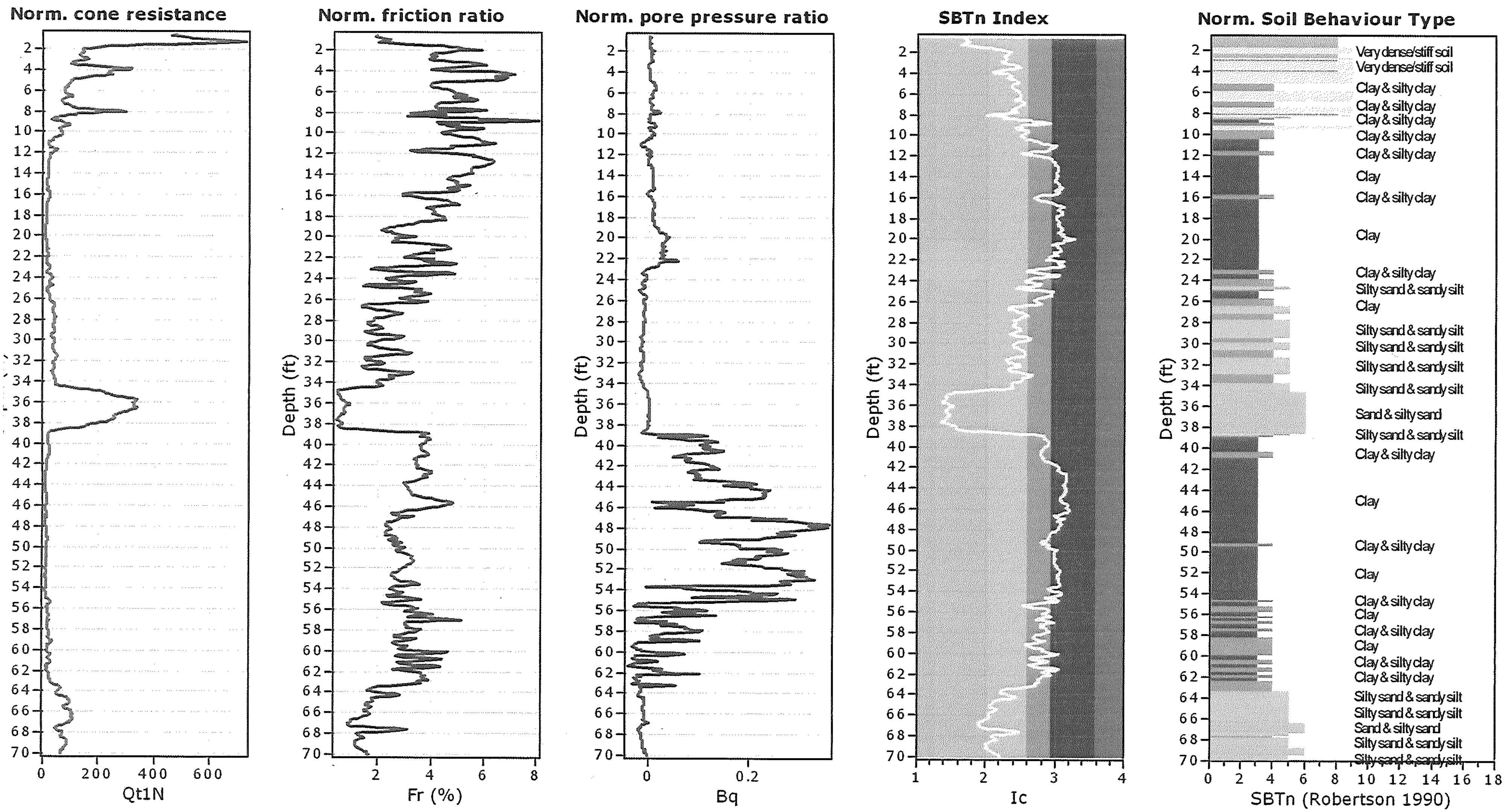
- ▨ 1. Sensitive fine grained
- ▨ 2. Organic material
- ▨ 3. Clay
- ▨ 4. Clay & silty clay
- ▨ 5. Clay & silty clay
- ▨ 6. Sandy silt & clayey silt
- ▨ 7. Silty sand & sandy silt
- ▨ 8. Sand & silty sand
- ▨ 9. Sand
- ▨ 10. Sand
- ☐ 11. Very dense/stiff soil
- ▨ 12. Very dense/stiff soil

PeT-IT v.1.6.0.68 - CPTU data presentation & interpretation software - Report created on: 6/3/2009, 1:53:40 PM
oject file: I:\Project\13000\13815.003\12000 CALC EVAL ANALYSES\Cliq\Penitencia Ck Levee Milpitas cpt.cpt

AMEC Geomatrix
2101 Webster St 12th Floor
Oakland CA 94612

roject: Penitencia Ck Levee Milpitas

CPT: CPT-04 - Total depth: 70.03 (ft)



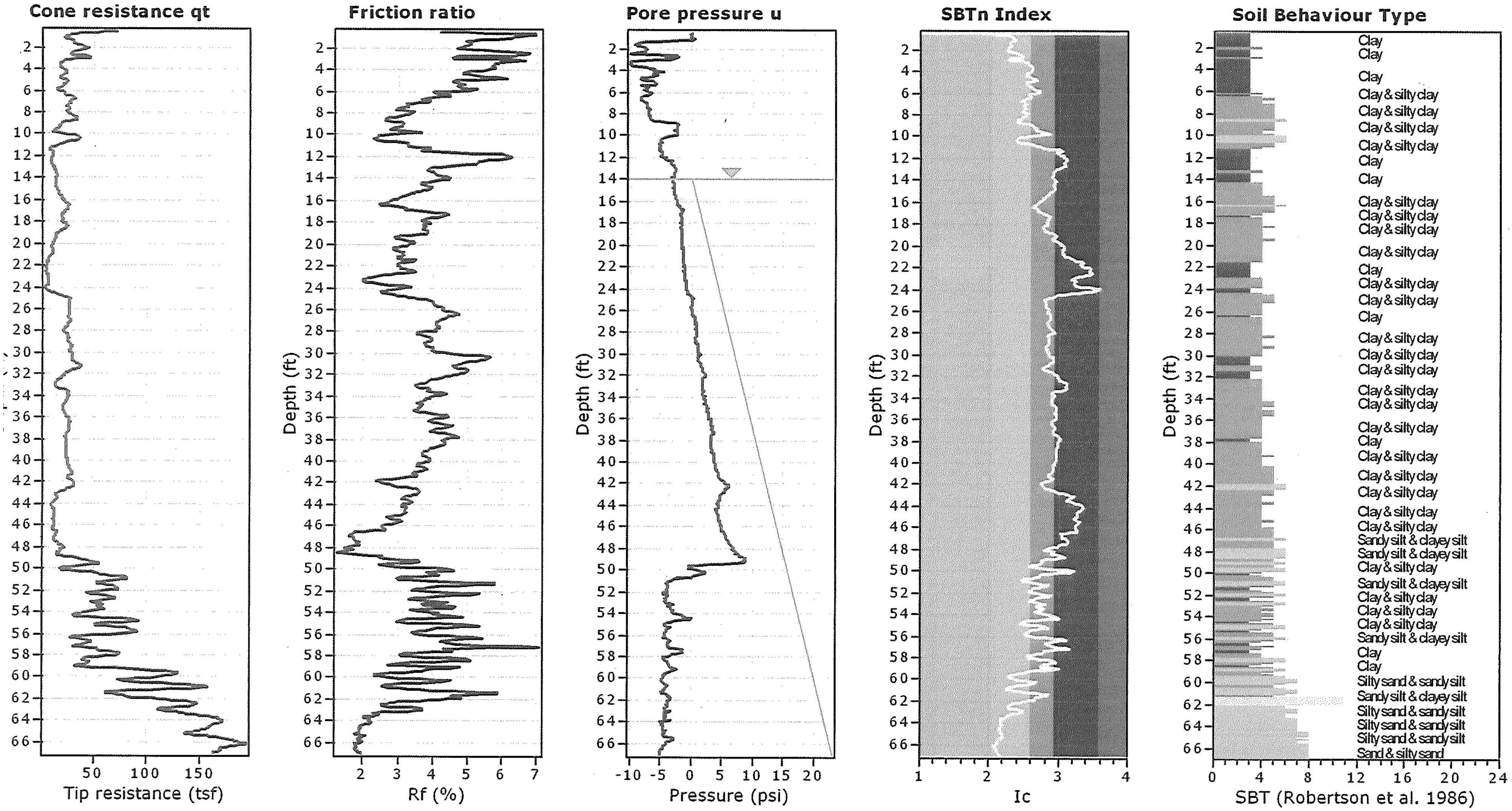
SBTn legend

▨ 1. Sensitive fine grained
▨ 2. Organic material
▨ 3. Clay to silty clay
▨ 4. Clayey silt to silty clay
▨ 5. Silty sand to sandy silt
▨ 6. Clean sand to silty sand
▨ 7. Gravely sand to sand
▨ 8. Very stiff sand to clayey sand
☐ 9. Very stiff fine grained

AMEC Geomatrix
2101 Webster St 12th Floor
Oakland CA 94612

roject: Penitencia Ck Levee Milpitas

CPT: CPT-05 - Total depth: 67.01 (ft)



SBT legend

■ 1. Sensitive fine grained	■ 4. Clay & silty clay	■ 7. Silty sand & sandy silt
■ 2. Organic material	■ 5. Clay & silty clay	■ 8. Sand & silty sand
■ 3. Clay	■ 6. Sandy silt & clayey silt	■ 9. Sand
■ 10. Sand	☐ 11. Very dense/stiff soil	■ 12. Very dense/stiff soil

PeT-IT v.1.6.0.68 - CPTU data presentation & interpretation software - Report created on: 6/3/2009, 1:53:41 PM
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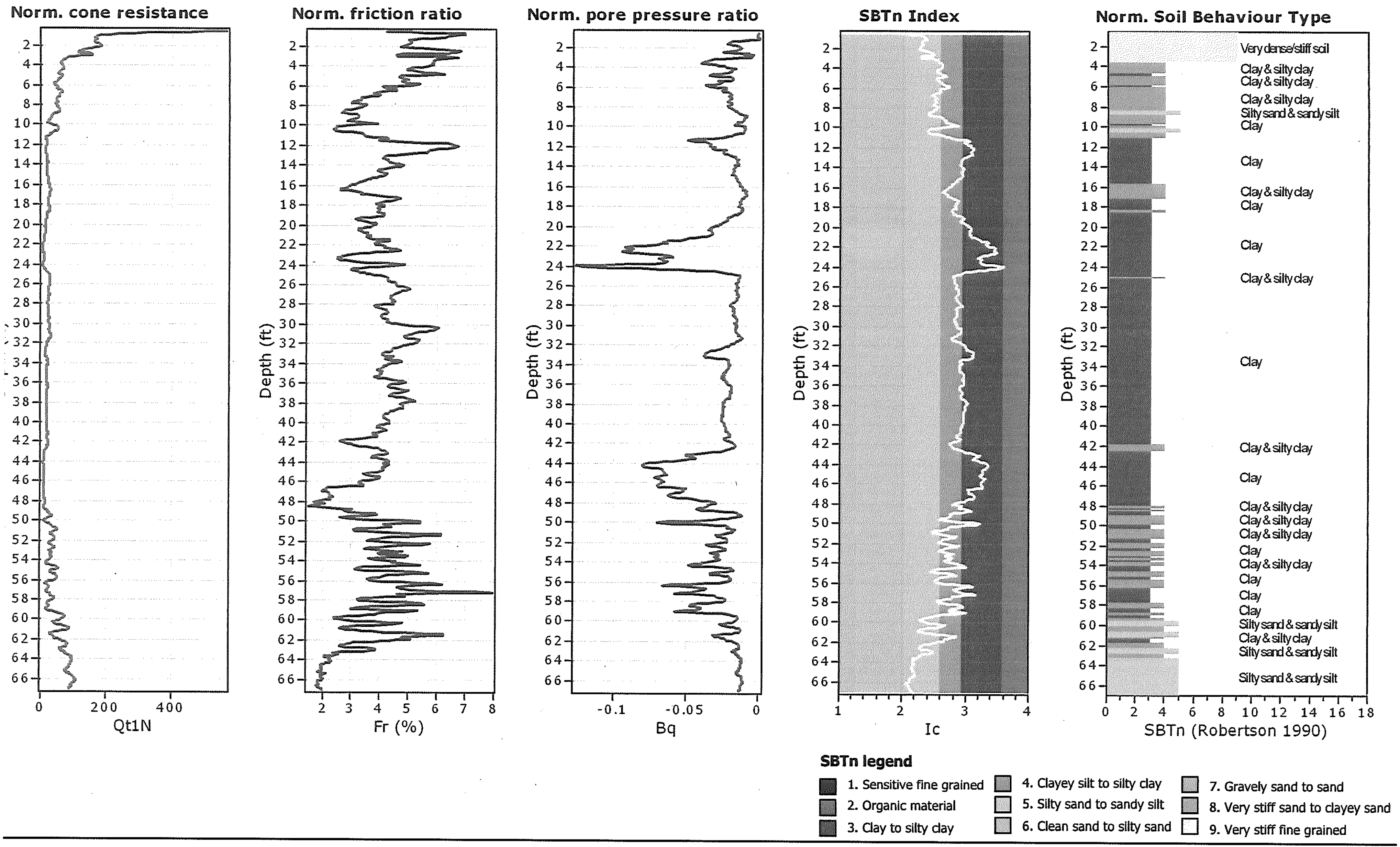
AMEC Geomatrix
2101 Webster St 12th Floor
Oakland CA 94612

roject: Penitencia Ck Levee Milpitas

CPT: CPT-05 - Total depth: 67.01 (ft)



APPENDIX D

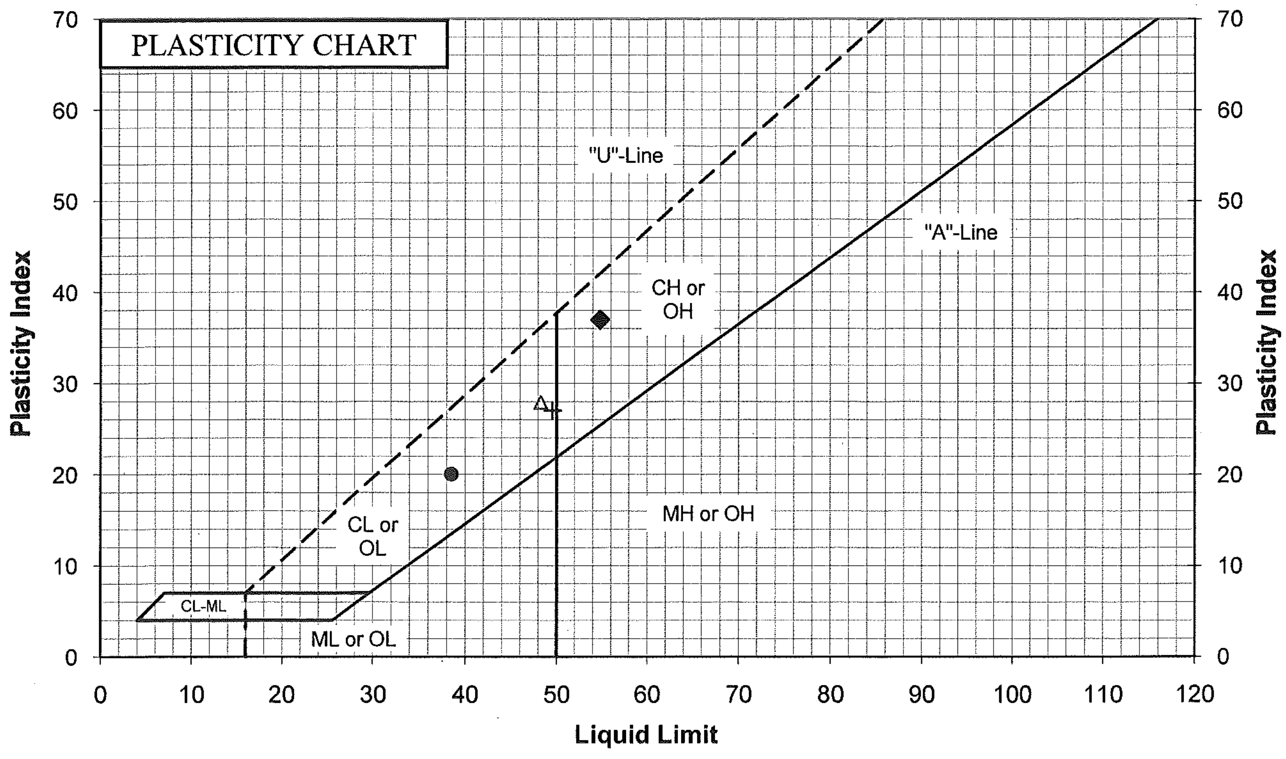
APPENDIX D

Geotechnical Laboratory Test Results

ATTERBERG LIMITS TEST RESULTS

PROJECT NAME	**SCVWD Lower Penitencia Creek Levee Recertification**	PROJECT No.	2227 E

DATE OF TEST	11/14/2008	11/17/2008	11/18/2008	11/17/2008	11/17/2008
KEY SYMBOL	♦		●	+	Δ
DRILL HOLE No.	B-1	B-1	B-1	B-2	B-2
DEPTH (ft)	9	13.5	36.5-37.5	1.5	5.5
NATURAL WATER CONTENT (%)	28	25	30	22	28
% Retained No. 40 SIEVE (Est.)	0	0	0	0	5
% PASSING No. 200 SIEVE	---	42	93	---	77
LIQUID LIMIT	55	---	38	50	48
PLASTIC LIMIT	18	Non-plastic	18	23	20
PLASTICITY INDEX	37	Non-plastic	20	27	28
CLASSIFICATION SYMBOL	CH	SM	CL	CH	CL



PACIFIC GEOTECHNICAL ENGINEERING

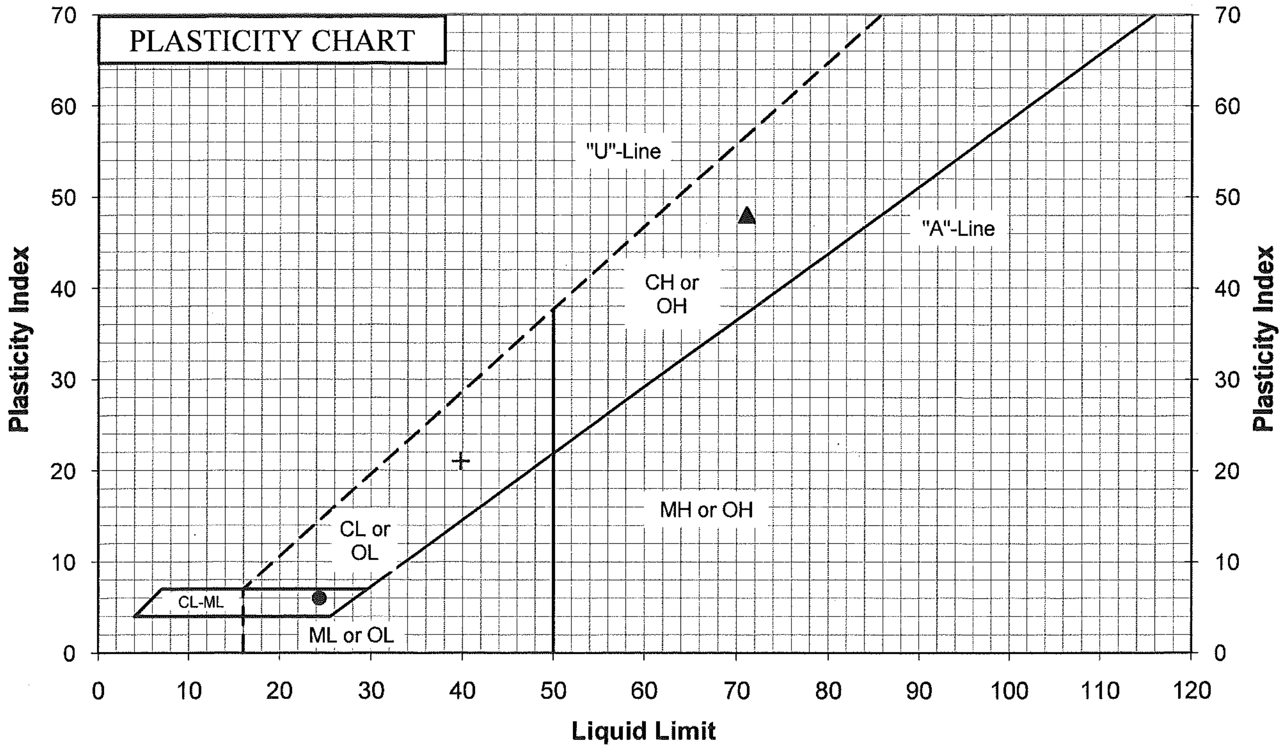
Rev030106

Figure B-1

ATTERBERG LIMITS TEST RESULTS

PROJECT NAME	**SCVWD Lower Penitencia Creek Levee Recertification**	PROJECT No.	2227 E

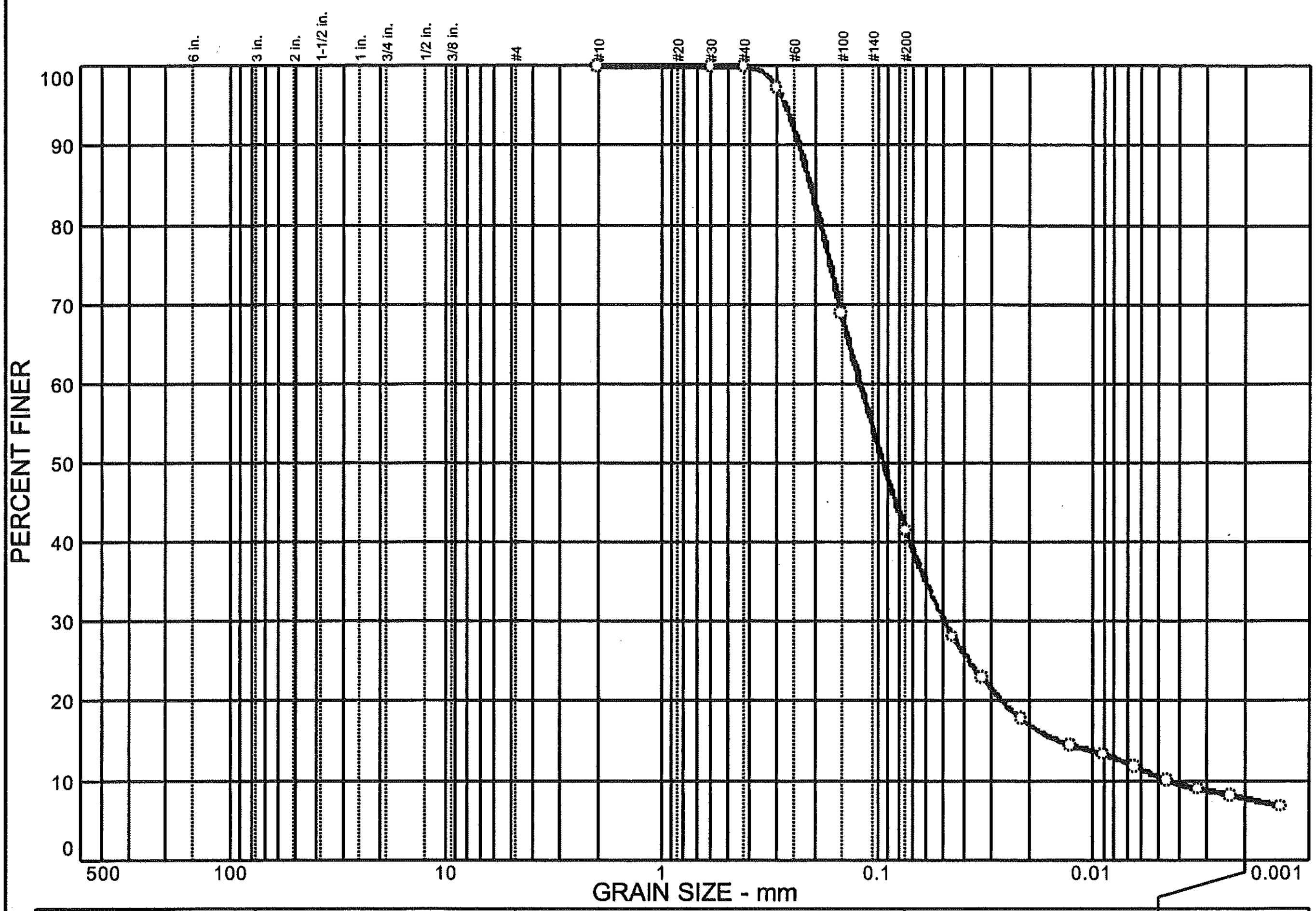
DATE OF TEST	11/26/2008	11/17/2008	11/18/2008	2/12/2009	
KEY SYMBOL		▲	●	+	
DRILL HOLE No.	B-2	B3	B3	B-2	
DEPTH (ft)	22 - 24	5.5	33.5-34.5	19	
NATURAL WATER CONTENT (%)	---	40	14	26	
% Retained No. 40 SIEVE (Est.)	0	5	0	4	
% PASSING No. 200 SIEVE	90	---	15	92	
LIQUID LIMIT	---	71	24	40	
PLASTIC LIMIT	Non-plastic	23	18	19	
PLASTICITY INDEX	Non-plastic	48	6	21	
CLASSIFICATION SYMBOL	ML	CH	SM/SC	CL	



PACIFIC GEOTECHNICAL ENGINEERING | **Figure B-2**

Rev030106

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	58.5	33.8	7.7

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#30	100.0		
#40	99.9		
#50	97.3		
#100	68.8		
#200	41.5		
0.0452 mm.	28.1		
0.0330 mm.	22.9		
0.0216 mm.	17.7		
0.0127 mm.	14.4		
0.0090 mm.	13.3		
0.0064 mm.	11.7		
0.0046 mm.	10.0		
0.0033 mm.	8.9		
0.0023 mm.	8.1		
0.0014 mm.	6.7		

* (no specification provided)

Soil Description

Gray Silty SAND

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.210 D_{60}= 0.123 D_{50}= 0.0959

D_{30}= 0.0494 D_{15}= 0.0146 D_{10}= 0.0046

C_u= 26.86 C_c= 4.31

Classification

USCS= AASHTO=

Remarks

Sample No.: **Source of Sample:** B-1 **Date:**

Location: **Elev./Depth:** 14'

COOPER TESTING LABORATORY

Client: Pacific Geotechnical Engineering

Project: Lower Penitencia Creek Levee - 2227E

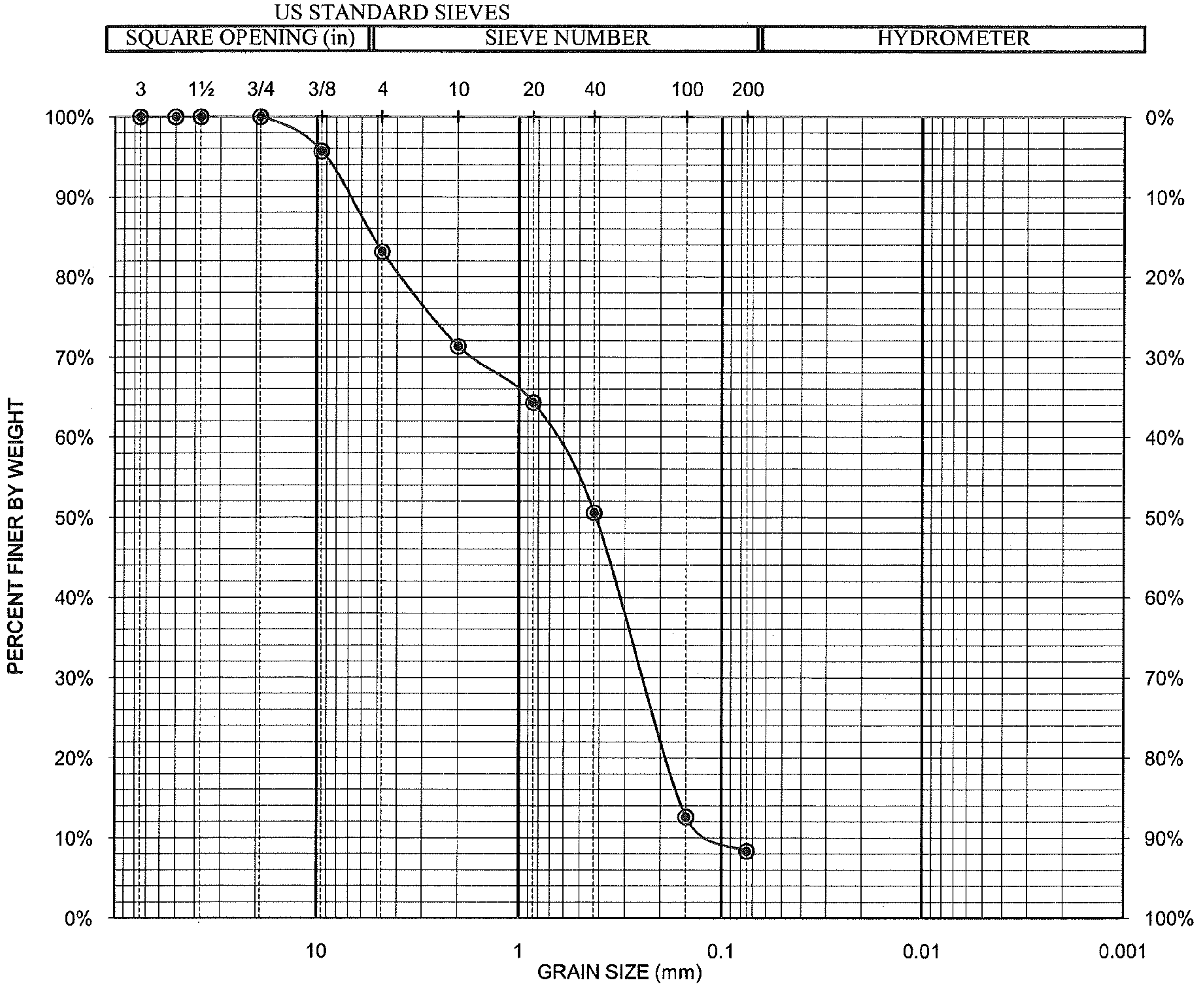
Project No: 226-174 **Figure** B-3

GRAIN SIZE TEST RESULTS

PROJECT NAME	SCVWD Lower Penitencia Creek Levee Recertification					**PROJECT No.**	2227 E
DRILL HOLE No.	B-1	**DEPTH (ft)**	28.5-29.5	**SAMPLE**	N/A	**DATE OF TEST**	11/25/2008

SOURCE/QUARRY: N/A

DESCRIPTION OF SOIL: POORLY GRADED SAND with GRAVEL and SILT (SP-SM): Dark gray (5Y 4/1)



	GRAVEL		SAND			SILT & CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	
COBBLES	16.9%		74.8%			8.3%

REMARKS:

PACIFIC GEOTECHNICAL ENGINEERING

Figure B-4

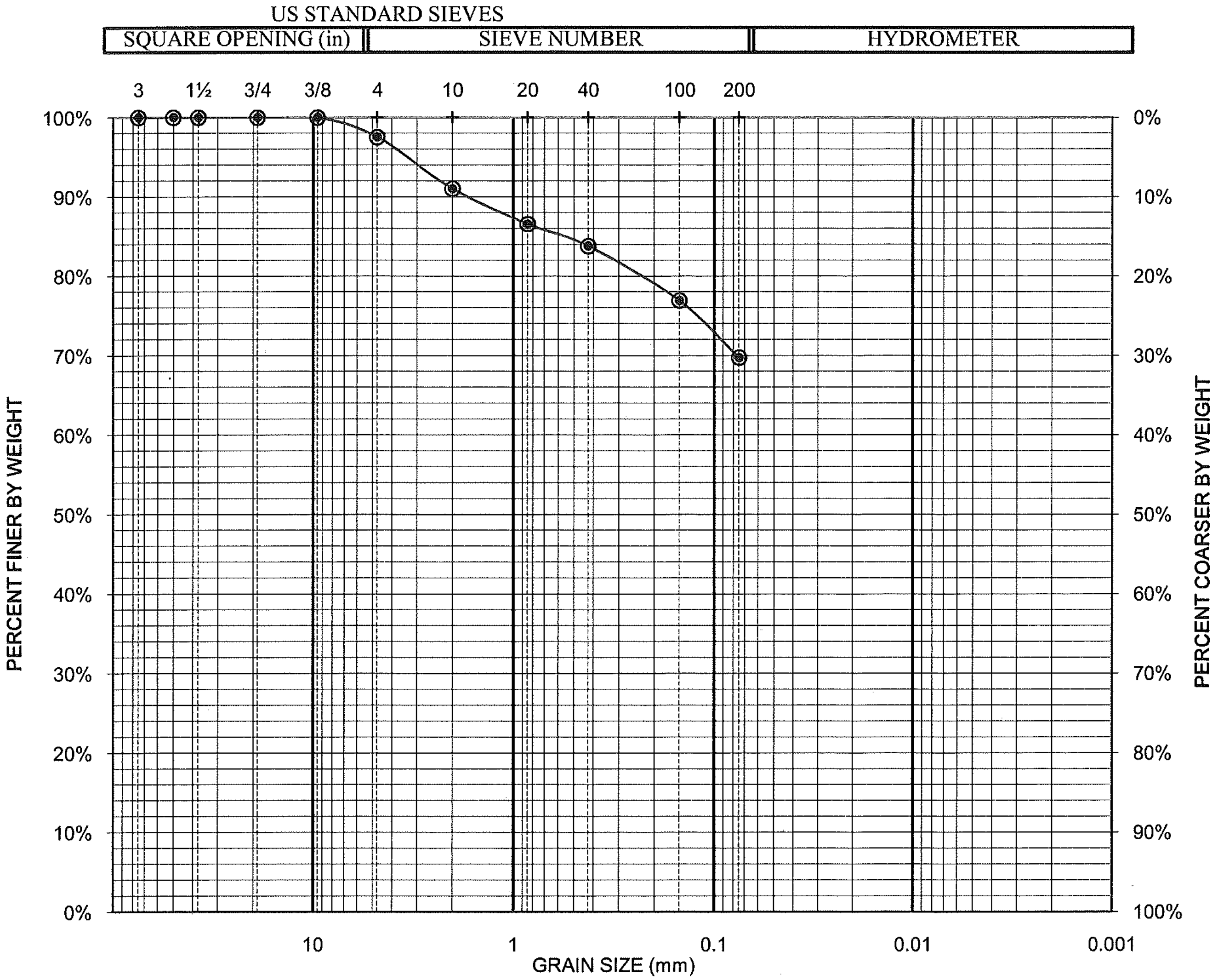
RevOct20070

GRAIN SIZE TEST RESULTS

PROJECT NAME	SCVWD Lower Penitencia Creek Levee Recertification					**PROJECT No.**	2227 E
DRILL HOLE No.	B-2	**DEPTH (ft)**	1.5	**SAMPLE**	N/A	**DATE OF TEST**	11/13/2008

SOURCE/QUARRY: N/A

DESCRIPTION OF SOIL: SANDY FAT CLAY: Very dark gray (10YR 3/1) to very dark brown (10YR 3/2)



	GRAVEL		SAND			SILT & CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	
COBBLES	GRAVEL		SAND			SILT & CLAY
	2.5%		27.8%			69.7%

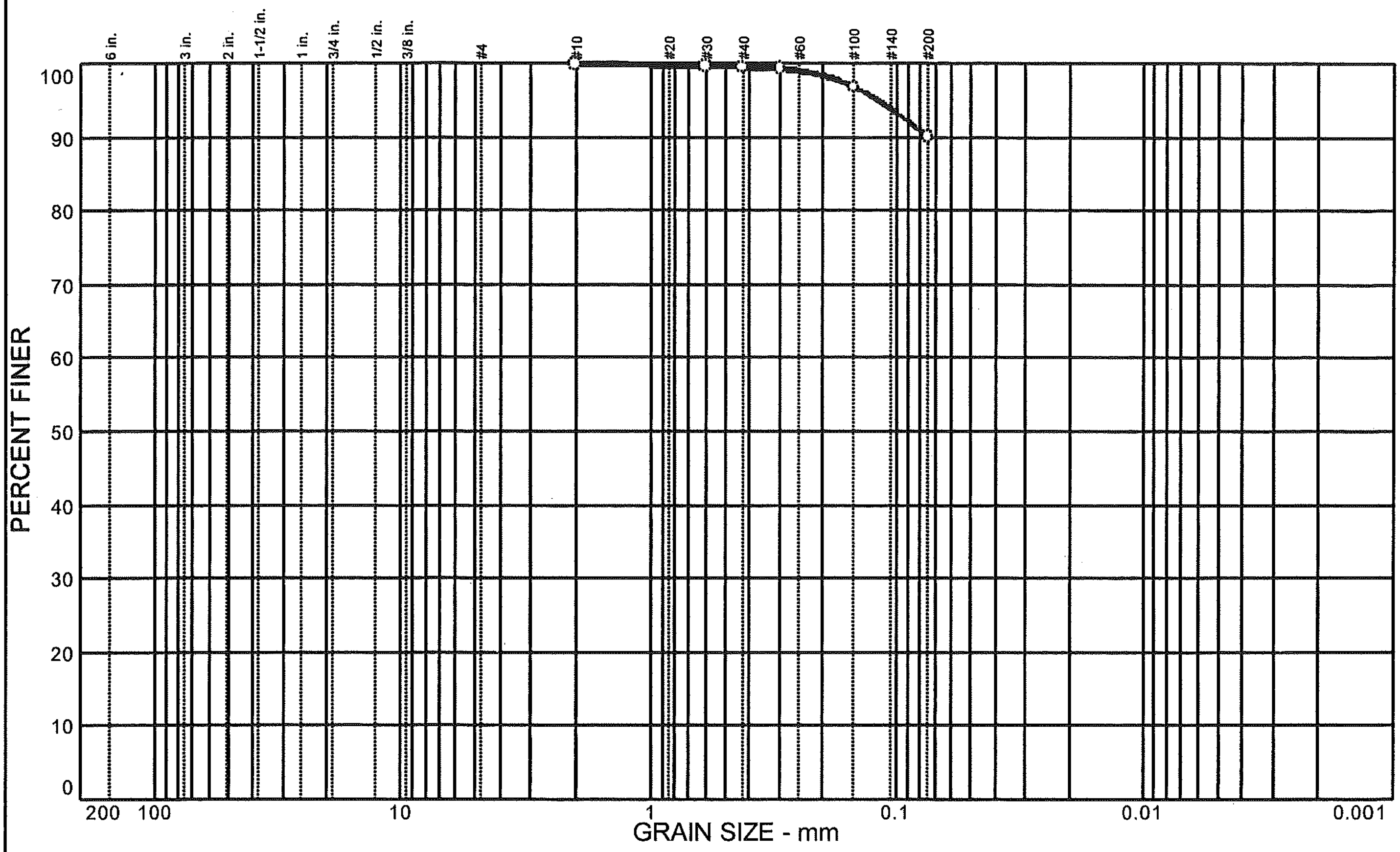
REMARKS:

RevOct20070

PACIFIC GEOTECHNICAL ENGINEERING

Figure B-5

Particle Size Distribution Report



	% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY	USCS	AASHTO	PL	LL
○			9.9	90.1		ML		NP	

SIEVE inches size	PERCENT FINER ○		
	GRAIN SIZE		
D60			
D30			
D10			
	COEFFICIENTS		
Cc			
Cu			

SIEVE number size	PERCENT FINER ○		
#10	100.0		
#30	99.6		
#40	99.5		
#50	99.3		
#100	96.9		
#200	90.1		

SOIL DESCRIPTION

○ Brown SILT

REMARKS:

○

○ Source: B-2

Elev./Depth: 22-24'

COOPER TESTING LABORATORY

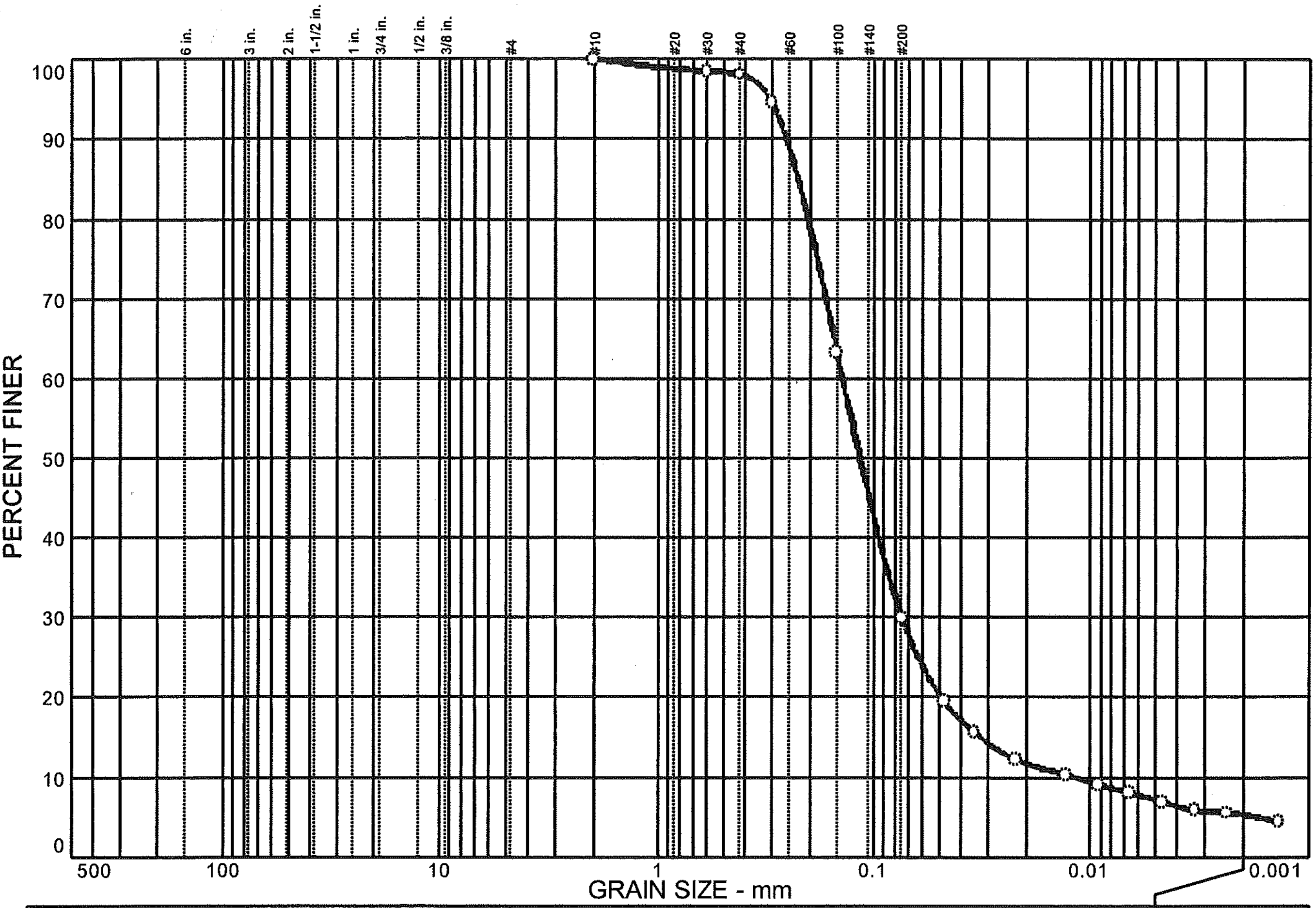
Client: Pacific Geotechnical Engineering

Project: Lower Penitencia Creek Levee - 2227E

Project No.: 226-174

Figure B-6

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	70.0	24.7	5.3

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#30	98.5		
#40	98.1		
#50	94.6		
#100	63.3		
#200	30.0		
0.0478 mm.	19.5		
0.0346 mm.	15.6		
0.0223 mm.	12.2		
0.0130 mm.	10.3		
0.0092 mm.	9.1		
0.0066 mm.	8.0		
0.0047 mm.	6.9		
0.0033 mm.	5.8		
0.0024 mm.	5.5		
0.0014 mm.	4.4		

* (no specification provided)

Soil Description

Gray Silty SAND

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.227 D_{60}= 0.141 D_{50}= 0.118
D_{30}= 0.0750 D_{15}= 0.0325 D_{10}= 0.0119
C_u= 11.91 C_c= 3.35

Classification

USCS= AASHTO=

Remarks

Sample No.: **Source of Sample:** B-3 **Date:**
Location: **Elev./Depth:** 14'

COOPER TESTING LABORATORY

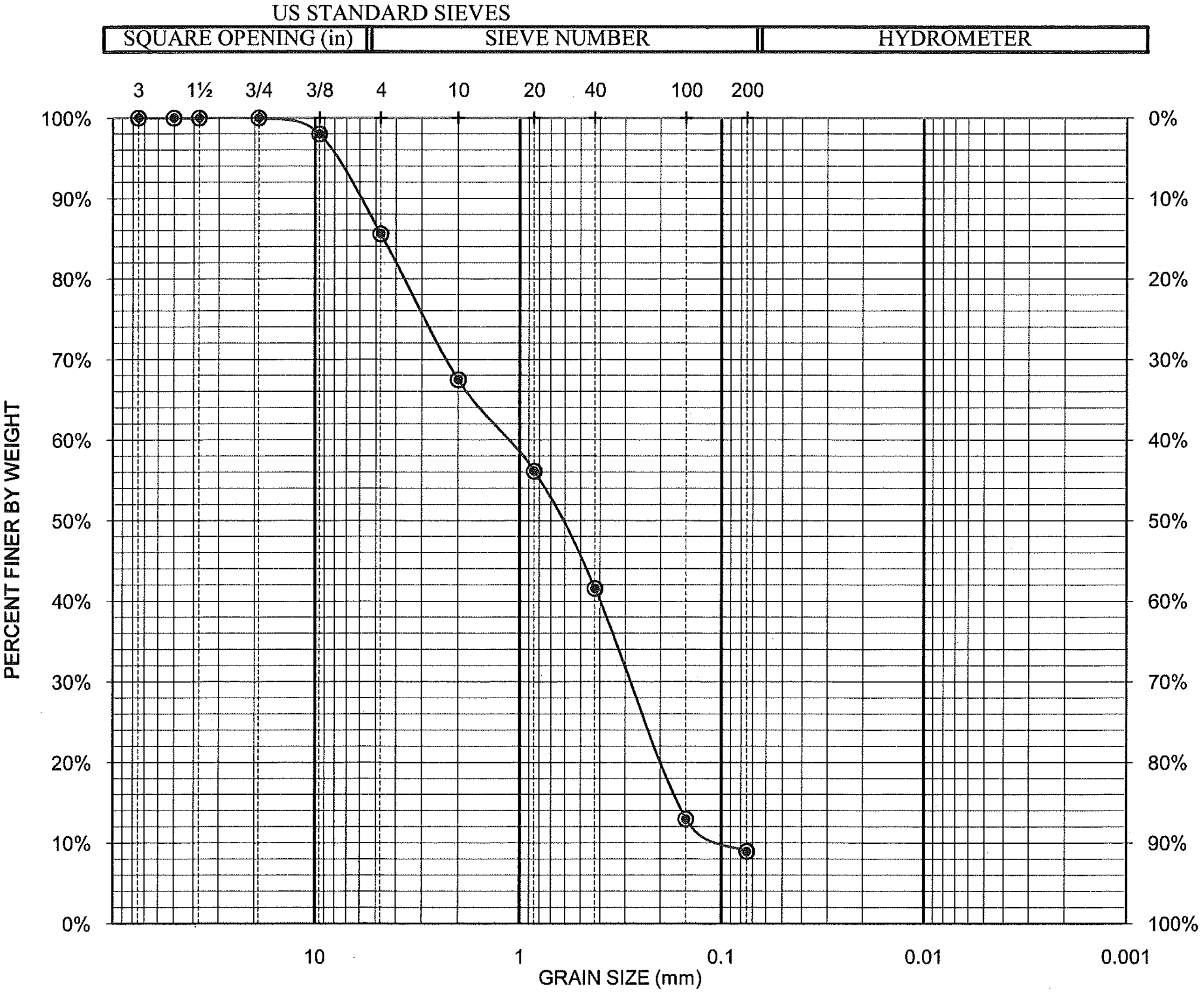
Client: Pacific Geotechnical Engineering
Project: Lower Penitencia Creek Levee - 2227E
Project No: 226-174 **Figure** B-7

GRAIN SIZE TEST RESULTS

PROJECT NAME	SCVWD Lower Penitencia Creek Levee Recertification					**PROJECT No.**	2227E
DRILL HOLE No.	B-3	**DEPTH (ft)**	28.5-29.5	**SAMPLE**	N/A	**DATE OF TEST**	11/25/2008

SOURCE/QUARRY: N/A

DESCRIPTION OF SOIL: POORLY GRADED SAND with CLAY (SP-SC): Dark gray (5Y4/1)



	COARSE	FINE	COARSE	MEDIUM	FINE	
COBBLES	GRAVEL		SAND			SILT & CLAY
	14.4%		76.7%			8.9%

REMARKS:

PACIFIC GEOTECHNICAL ENGINEERING

RevOct20070

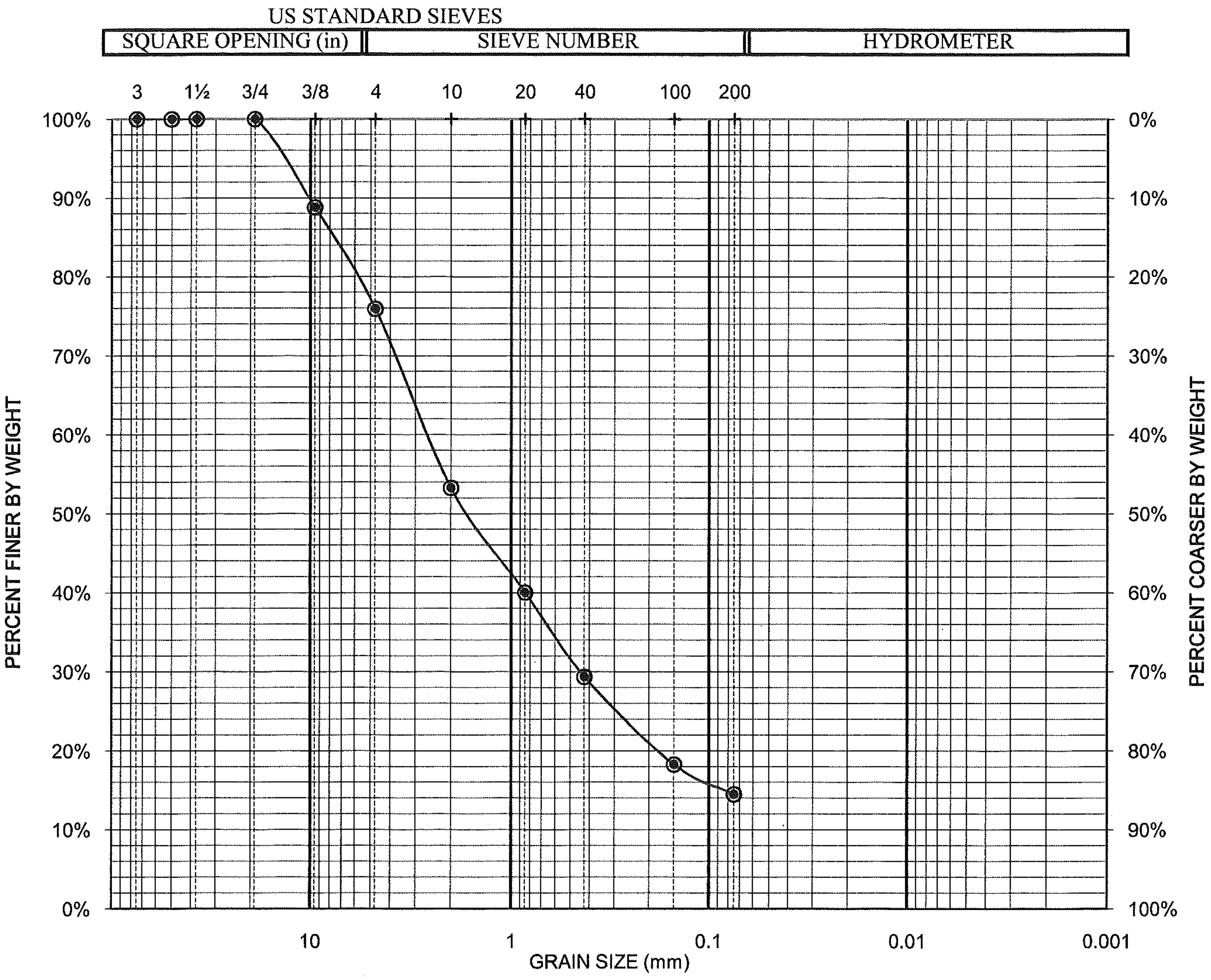
Figure **B-8**

GRAIN SIZE TEST RESULTS

PROJECT NAME	SCVWD Lower Penitencia Creek Levee Recertification					**PROJECT No.**	2227 E
DRILL HOLE No.	B-3	**DEPTH (ft)**	33.5-34.5	**SAMPLE**	N/A	**DATE OF TEST**	11/14/2008

SOURCE/QUARRY: N/A

DESCRIPTION OF SOIL: SILTY SAND with GRAVEL: Bluish black (gley2 2.5/10B)



	GRAVEL		SAND			SILT & CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	
COBBLES	GRAVEL		SAND			SILT & CLAY
	24.1%		61.4%			14.5%

REMARKS:

PACIFIC GEOTECHNICAL ENGINEERING

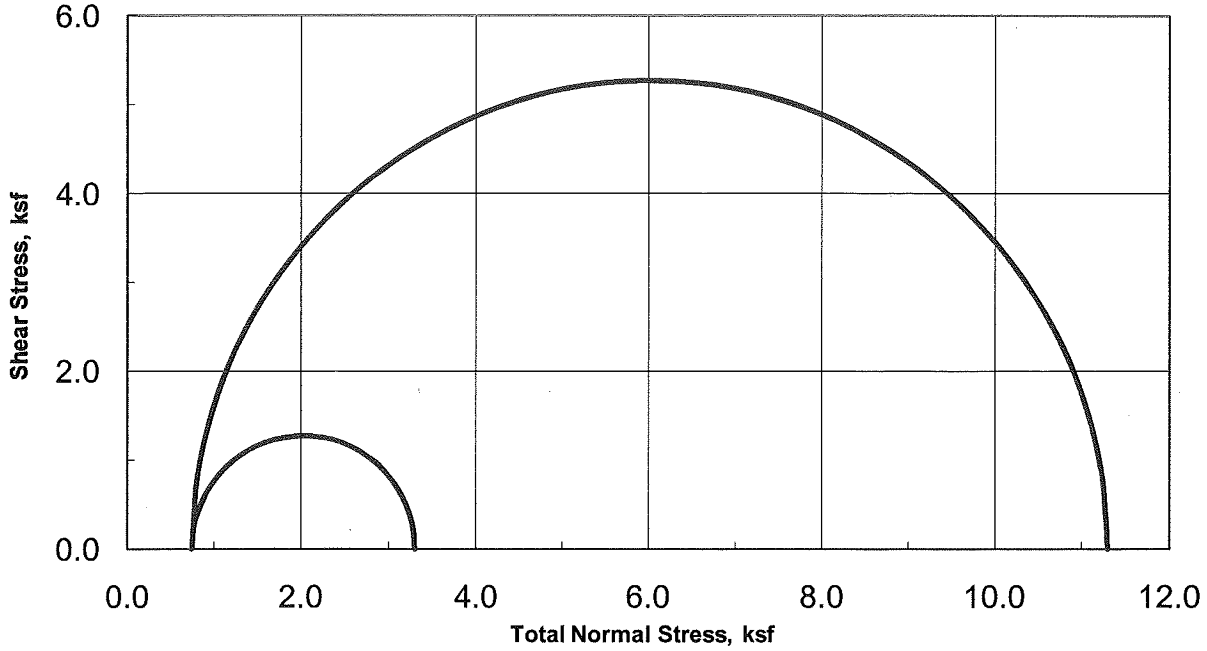
Figure **B-9**

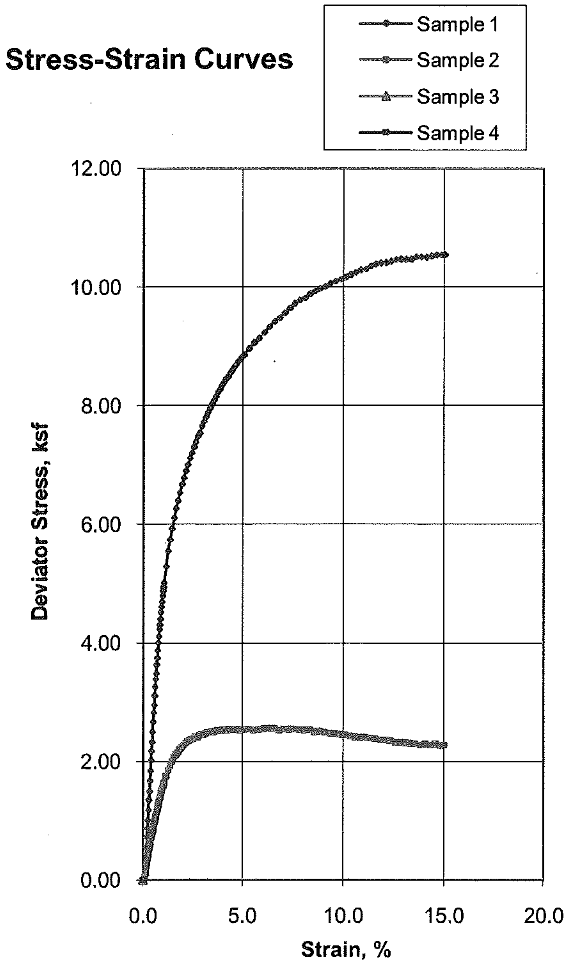
RevOct20070

COOPER TESTING LABORATORY

Unconsolidated-Undrained Triaxial Test

ASTM D-2850





Sample Data				
	1	2	3	4
Moisture %	22.9	43.9		
Dry Den,pcf	103.0	76.9		
Void Ratio	0.636	1.192		
Saturation %	97.0	99.5		
Height in	4.98	5.00		
Diameter in	2.40	2.41		
Cell psi	5.2	5.2		
Strain %	14.60	6.10		
Deviator, ksf	10.542	2.553		
Rate %/min	1.00	1.00		
in/min	0.050	0.050		
Job No.:	**226-174**			
Client:	**Pacific Geotechnical Engineering**			
Project:	**2227E**			
Boring:	B-2	B-3		
Sample:				
Depth ft:	6.0	6.0		

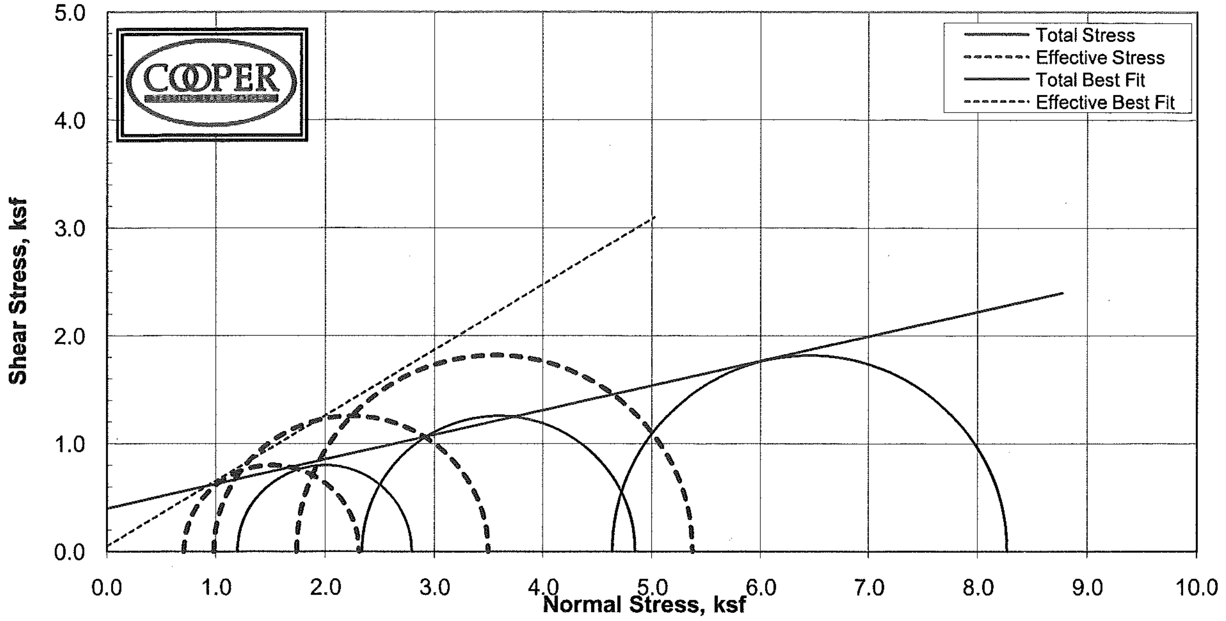
Visual Soil Description	
Sample #	
1	Dark Gray Sandy CLAY
2	Mottled Gray & Brown CLAY w/ Sand
3	
4	

Remarks:

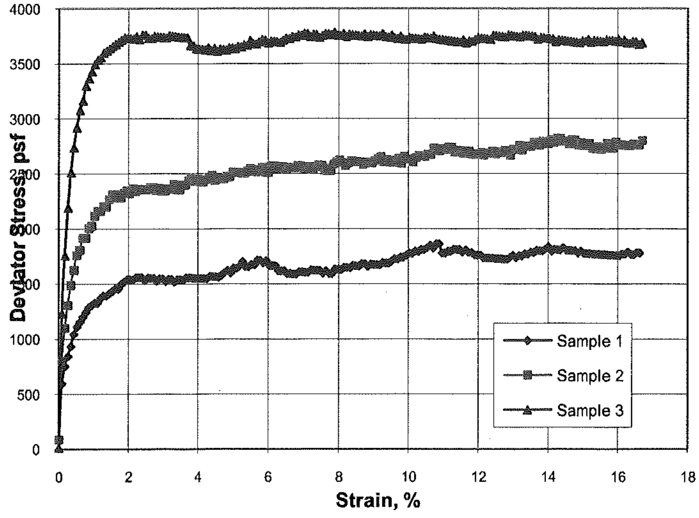
Figure B-10

Triaxial Consolidated Undrained

(ASTM D4767)



Stress-Strain Response



Sample:	1	2	3	4
MC, %	32.0	28.3	23.6	
Dry Dens, pcf	90.3	94.6	101.5	
Sat. %	99.8	97.8	96.6	
Void Ratio	0.866	0.781	0.660	
Diameter in	2.86	2.86	2.86	
Height, in	6.01	6.00	6.00	
	Final			
MC, %	31.0	26.4	22.2	
Dry Dens, pcf	91.7	98.4	105.3	
Sat. %	100.0	100.0	100.0	
Void Ratio	0.837	0.713	0.599	
Diameter, in	2.85	2.82	2.82	
Height, in	5.98	5.94	5.93	
Cell, psi	46.5	54.5	70.4	
BP, psi	38.2	38.3	38.2	
	Effective Stresses At:			
Strain, %	5.0	5.0	5.0	
Deviator ksf	1.600	2.514	3.639	
Excess PP	0.487	1.354	2.900	
Sigma 1	2.307	3.493	5.371	
Sigma 3	0.707	0.979	1.732	
P, ksf	1.507	2.236	3.552	
Q, ksf	0.800	1.257	1.820	
Stress Ratio	3.262	3.567	3.101	
Rate in/min	0.001	0.001	0.001	
Total C	0.4	ksf		
Total Phi	12.8	Degrees		
Eff. C	0.0	ksf		
Eff. Phi	31.3	Degrees		

Job No.: 226-174 **Date:** 11/26/2008

Client: Pacific Geotechnical Engineerin BY:MD/DC

Project: 2227E

Sample 1) B-2 @ 22-24' Brown SILT

Sample 2) B-2 @ 22-24' Brown SILT

Sample 3) B-2 @ 22-24' Brown SILT

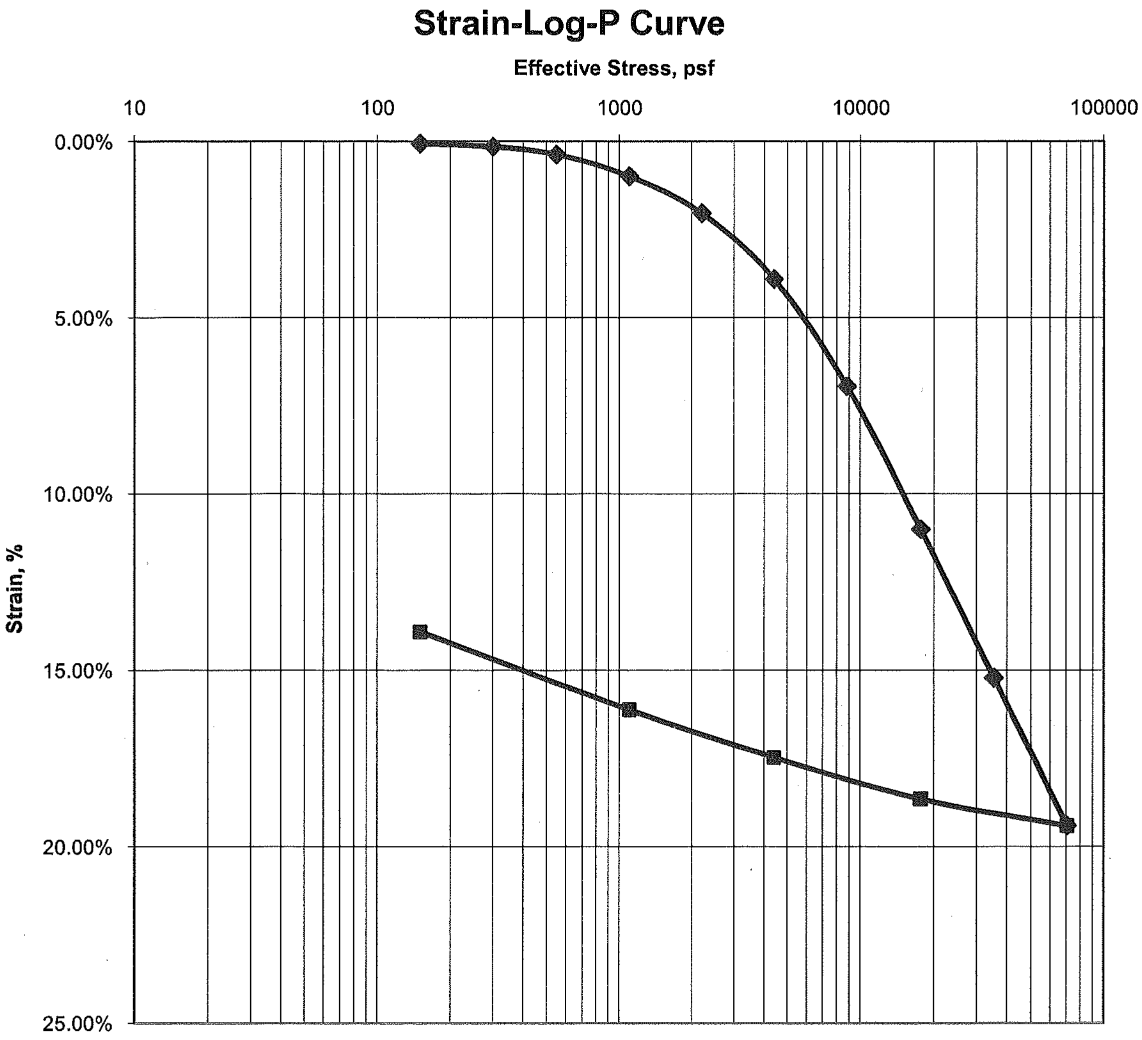
Sample 4)

REMARKS: Strengths picked at 5% strain.

Figure B-11

Consolidation Test
ASTM D2435

Job No.:	226-174	**Boring:**	B-2	**Run By:**	MD
Client:	Pacific Geotechnical Eng	**Sample:**		**Reduced:**	PJ
Project:	2227E	**Depth, ft.:**	22-24	**Checked:**	PJ/DC
Soil Type:	Brown SILT			**Date:**	11/26/2008

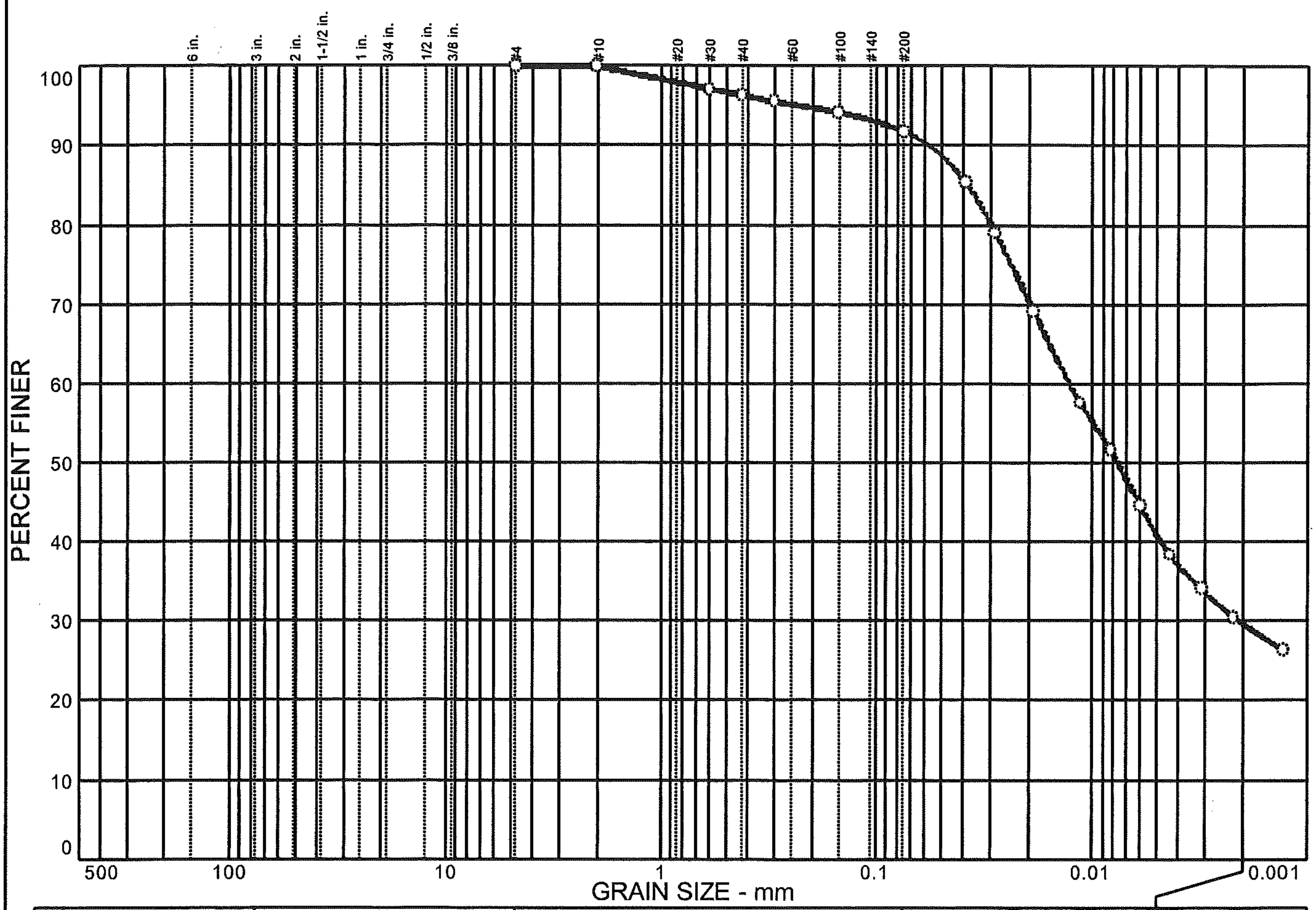


Ass. Gs = 2.75	**Initial**	**Final**
Moisture %:	27.8	20.7
Dry Density, pcf:	95.7	109.5
Void Ratio:	0.794	0.567
% Saturation:	96.4	100

Remarks:

Figure B-12

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	8.3	62.0	29.7

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	99.9		
#30	97.1		
#40	96.3		
#50	95.5		
#100	94.1		
#200	91.7		
0.0389 mm.	85.3		
0.0284 mm.	78.9		
0.0188 mm.	69.0		
0.0114 mm.	57.5		
0.0082 mm.	51.5		
0.0060 mm.	44.5		
0.0043 mm.	38.3		
0.0031 mm.	34.0		
0.0022 mm.	30.5		
0.0013 mm.	26.3		

* (no specification provided)

Soil Description

Brown CLAY

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.0383 D_{60}= 0.0129 D_{50}= 0.0077
D_{30}= 0.0021 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= AASHTO=

Remarks

Sample No.: **Source of Sample:** B2 **Date:**
Location: **Elev./Depth:** 19.0'

COOPER TESTING LABORATORY

Client: Pacific Geotechnical Engineering
Project: SCVWD FEMA Lower Penitencia Creek - 2227E
Project No: 226-176 **Figure**

APPENDIX E

APPENDIX E

Summary of Liquefaction Analysis Results

ATTACHMENT E.1 SPT Liquefaction Results

ATTACHMENT E.2 CLiq Liquefaction Results

APPENDIX E

LIQUEFACTION ANALYSES
Lower Penitencia Creek Levee Recertification
Milpitas, California

Liquefaction susceptibility has been evaluated for design level ground shaking, defined as a peak ground acceleration (PGA) that has probability of exceedance of 10 percent in 50 years. Based on this definition, we used a PGA of 0.67g, with a typical (modal) Moment Magnitude of 6.9.

Liquefaction susceptibility of materials encountered in the borings has been evaluated using a spreadsheet developed by AMEC Geomatrix. We have generally followed the methods of Youd et al. (2001). For hammer energy, we have conservatively assumed a 60 percent efficiency (matching the standard value for rope-and-cathead testing). The method for analyzing the amount of settlement during a seismic event of saturated soils follows Tokimatsu and Seed (1984).

Liquefaction susceptibility of materials encountered in the CPTs has been evaluated by the computer program CLiq (Version 1.3, 2006, revised 2009), developed by GeoLogismiki in collaboration with Peter Robertson of Gregg Drilling. An explanation of the calculation methods used by CLiq follows the spreadsheets for the borings, and the results of the complete analyses follow the explanation of the method.

ATTACHMENT E.1

SPT Liquefaction Results

STANDARD PENETRATION TEST DATA

amec

Parameter	Value	Units	Notes
Magnitude =	6.9		
a =	0.67	g	g=gravity
o1 = other w/ efficiency	60%	%	
o2 = other w/efficiency	100%	%	

Borehole diameter 4 in 100

Key:	
Entered	Value
Calculated	Value
Assumed	Value
	Auger boring unreliable for liquefaction anaylsis

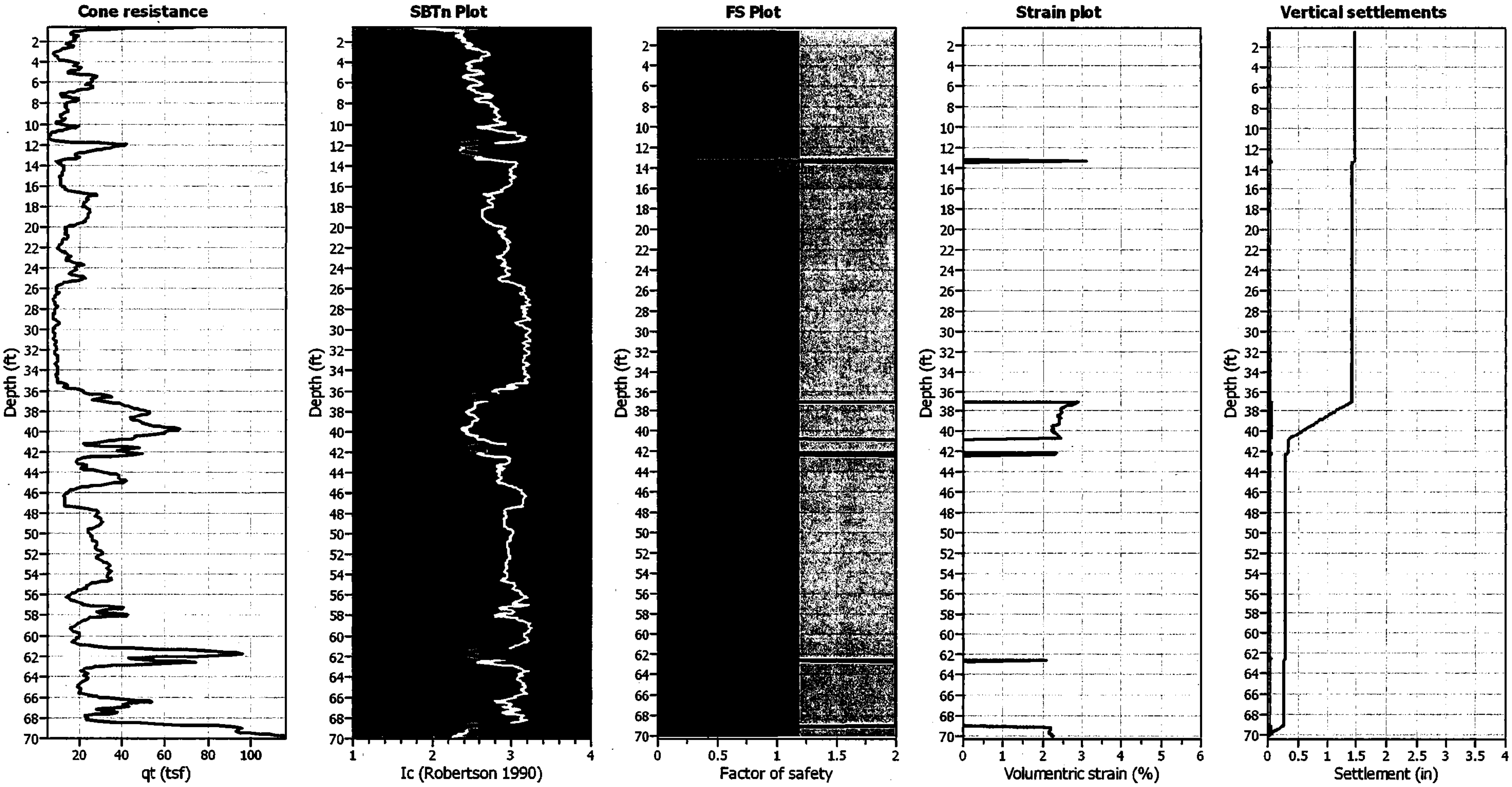
Seismic Groundwater (ft) = 4

Boring Number	Sample Depth (ft)	N	Sampler Type	Sampler Designed to Have Liners? (y/n)	Hammer Configuration	C_R	C_s	C_S	C_E	C_B	N_{60}	gwt (ft)	γ (pcf)	σ (psf)	σ' (psf)	C_N	$(N_1)_{60}$	USCS	FC (%)		Alpha	Beta	(N1)60-cs	CRR7.5 CS	CRR7.5	rd	CSR	MSF	K_s	CSR7.5	FS
B-1	1.5	13	m	n	o2	0.75	0.65	1.0	1.00	1.00	6	38	125	187.5	188	1.7	11	CH	50		5.00	1.20	18	0.19	0.12	1.00	0.43	1.24	1.00	0.35	-
B-1	4.5	11	m	n	o2	0.75	0.65	1.0	1.00	1.00	5	38	125	562.5	563	1.7	9	CH	50		5.00	1.20	16	0.17	0.10	0.99	0.43	1.24	1.00	0.35	-
B-1	6.5	13	m	n	o2	0.75	0.65	1.0	1.00	1.00	6	38	125	812.5	813	1.6	10	CH	50		5.00	1.20	17	0.18	0.11	0.98	0.43	1.24	1.00	0.35	-
B-1	9.5	17	m	n	o2	0.75	0.65	1.0	1.00	1.00	8	38	125	1187.5	1188	1.3	11	CH	50		5.00	1.20	18	0.19	0.12	0.98	0.43	1.24	1.00	0.34	-
B-1	14.5	6	m	n	o2	0.85	0.65	1.0	1.00	1.00	3	38	125	1812.5	1813	1.1	3	SM	42		5.00	1.20	9	0.10	0.06	0.97	0.42	1.24	1.00	0.34	0.30
B-1	19.5	8	SPT	y	o2	0.85	1	1.1	1.00	1.00	8	38	125	2437.5	2438	0.9	7	SM	21		3.78	1.09	11	0.13	0.09	0.95	0.42	1.24	0.97	0.35	0.36
B-1	24.5	14	SPT	y	o2	0.95	1	1.2	1.00	1.00	16	38	125	3062.5	3063	0.8	13	SM	32		4.83	1.17	20	0.22	0.14	0.94	0.41	1.24	0.91	0.36	0.59
B-1	29.5	19	SPT	y	o2	0.95	1	1.2	1.00	1.00	22	38	125	3687.5	3688	0.7	16	SP-SM	8		0.30	1.01	17	0.18	0.17	0.93	0.41	1.24	0.87	0.38	0.47
B-1	34.5	21	SPT	y	o2	1	1	1.2	1.00	1.00	26	38	125	4312.5	4313	0.7	18	SC	30		4.71	1.15	25	0.30	0.19	0.89	0.39	1.24	0.84	0.38	0.80
B-1	37.5	16	SPT	y	o2	1	1	1.2	1.00	1.00	19	38	125	4687.5	4688	0.7	12	CL	93		5.00	1.20	19	0.21	0.13	0.87	0.38	1.24	0.82	0.37	-
B-2	2.5	26	m	n	o2	0.75	0.65	1.0	1.00	1.00	13	30	125	312.5	313	1.7	22	CH	70		5.00	1.20	31	-	0.24	0.99	0.43	1.24	1.00	0.35	-
B-2	4.5	34	m	n	o2	0.75	0.65	1.0	1.00	1.00	17	30	125	562.5	563	1.7	28	CH	50		5.00	1.20	39	-	0.37	0.99	0.43	1.24	1.00	0.35	-
B-2	6.5	44	m	n	o2	0.75	0.65	1.0	1.00	1.00	22	30	125	812.5	813	1.6	34	CL	77		5.00	1.20	46	-	#DIV/0!	0.98	0.43	1.24	1.00	0.35	-
B-2	9.5	54	m	n	o2	0.75	0.65	1.0	1.00	1.00	26	30	125	1187.5	1188	1.3	34	CL	50		5.00	1.20	46	-	#DIV/0!	0.98	0.43	1.24	1.00	0.34	-
B-2	14.5	14	m	n	o2	0.85	0.65	1.0	1.00	1.00	8	30	125	1812.5	1813	1.1	8	CH	50		5.00	1.20	15	0.16	0.10	0.97	0.42	1.24	1.00	0.34	-
B-2	19.5	14	m	n	o2	0.85	0.65	1.0	1.00	1.00	8	30	125	2437.5	2438	0.9	7	ML	92		5.00	1.20	13	0.14	0.09	0.95	0.42	1.24	0.97	0.35	0.42
B-2	29.5	30	m	n	o2	0.95	0.65	1.0	1.00	1.00	19	30	125	3687.5	3688	0.7	14	CL	98		5.00	1.20	22	0.24	0.15	0.93	0.41	1.24	0.87	0.38	-
DH-1	2.5	21	m	n	o1	0.75	0.65	1.0	0.60	1.00	6	10	125	312.5	313	1.7	10	SM	23		4.06	1.10	15	0.16	0.11	0.99	0.43	1.24	1.00	0.35	-
DH-1	6.5	20	m	n	o1	0.75	0.65	1.0	0.60	1.00	6	10	125	812.5	813	1.6	9	ML	50		5.00	1.20	16	0.17	0.10	0.98	0.43	1.24	1.00	0.35	0.49
DH-1	16.5	31	m	n	o1	0.85	0.65	1.0	0.60	1.00	10	10	125	2062.5	1657	1.1	11	CH	50		5.00	1.20	18	0.19	0.12	0.96	0.52	1.24	1.00	0.42	-
DH-1	21.5	14	m	n	o1	0.95	0.65	1.0	0.60	1.00	5	10	125	2687.5	1970	1.0	5	CL	50		5.00	1.20	11	0.12	0.07	0.95	0.56	1.24	1.00	0.46	-
DH-1	26.5	18	m	n	o1	0.95	0.65	1.0	0.60	1.00	7	10	125	3312.5	2283	0.9	6	CH	50		5.00	1.20	12	0.13	0.08	0.94	0.59	1.24	0.98	0.49	-
DH-1	31.5	69	m	n	o1	0.95	0.65	1.0	0.60	1.00	26	10	125	3937.5	2596	0.9	22	ML	50		5.00	1.20	31	-	0.24	0.92	0.61	1.24	0.95	0.51	-
DH-1	36.5	83	m	n	o1	1	0.65	1.0	0.60	1.00	32	10	125	4562.5	2909	0.8	27	CL	50		5.00	1.20	37	-	0.34	0.88	0.60	1.24	0.92	0.52	-
DH-1	41.5	20	m	n	o1	1	0.65	1.0	0.60	1.00	8	10	125	5187.5	3222	0.8	6	CL	50		5.00	1.20	12	0.13	0.08	0.84	0.59	1.24	0.90	0.53	-
DH-1	46.5	28	m	n	o1	1	0.65	1.0	0.60	1.00	11	10	125	5812.5	3535	0.8	8	ML-CL	50		5.00	1.20	15	0.16	0.10	0.80	0.57	1.24	0.88	0.52	0.30
DH-1	51.0	70	m	n	o1	1	0.65	1.0	0.60	1.00	27	10	125	6375	3817	0.7	20	ML-CL	50		5.00	1.20	29	0.41	0.22	0.76	0.55	1.24	0.86	0.52	0.79
DH-1	56.5	20	m	n	o1	1	0.65	1.0	0.60	1.00	8	10	125	7062.5	4161	0.7	5	ML	50		5.00	1.20	11	0.12	0.07	0.71	0.53	1.24	0.84	0.50	0.24
DH-1	61.5	56	m	n	o1	1	0.65	1.0	0.60	1.00	22	10	125	7687.5	4474	0.7	15	ML	50		5.00	1.20	23	0.26	0.16	0.67	0.50	1.24	0.83	0.49	0.52
DH-1	66.0	67	m	n	o1	1	0.65	1.0	0.60	1.00	26	10	125	8250	4756	0.6	17	CL	50		5.00	1.20	25	0.30	0.18	0.64	0.48	1.24	0.82	0.48	-
DH-1	71.5	53	m	n	o1	1	0.65	1.0	0.60	1.00	21	10	125	8937.5	5100	0.6	13	SM-ML	50		5.00	1.20	21	0.22	0.14	0.59	0.45	1.24	0.80	0.45	0.49
DH-1	74.0	44	m	n	o1	1	0.65	1.0	0.60	1.00	17	10	125	9250	5256	0.6	11	CL	50		5.00	1.20	18	0.19	0.12	0.57	0.44	1.24	0.80	0.44	-
B-3	2.5	16	m	n	o2	0.75	0.65	1.0	1.00	1.00	8	35	125	312.5	313	1.7	13	CH	50		5.00	1.20	21	0.22	0.14	0.99	0.43	1.24	1.00	0.35	-
B-3	4.5	15	m	n	o2	0.75	0.65	1.0	1.00	1.00	7	35	125	562.5	563	1.7	12	CH	50		5.00	1.20	19	0.21	0.13	0.99	0.43	1.24	1.00	0.35	-
B-3	6.5	9	m	n	o2	0.75	0.65	1.0	1.00	1.00	4	35	125	812.5	813	1.6	7	CH	50		5.00	1.20	13	0.14	0.09	0.98	0.43	1.24	1.00	0.35	-
B-3	9.5	16	m	n	o2	0.75	0.65	1.0	1.00	1.00	8	35	125	1187.5	1188	1.3	10	CH	50		5.00	1.20	17	0.18	0.11	0.98	0.43	1.24	1.00	0.34	-
B-3	14.5	13	m	n	o2	0.85	0.65	1.0	1.00	1.00	7	35	125	1812.5	1813	1.1	8	SM	30		4.71	1.15	14	0.15	0.10	0.97	0.42	1.24	1.00	0.34	0.44
B-3	19.5	15	SPT	y	o2	0.85	1	1.2	1.00	1.00	15	35	125	2437.5	2438	0.9	14	SM	23		4.06	1.10	19	0.21	0.15	0.95	0.42	1.24	0.97	0.35	0.60
B-3	24.5	19	SPT	y	o2	0.95	1	1.2	1.00	1.00	22	35	125	3062.5	3063	0.8	18	SM	14		2.20	1.04	21	0.23	0.19	0.94	0.41	1.24	0.91	0.36	0.63
B-3	29.5	25	SPT	y	o2	0.95	1	1.3	1.00	1.00	30	35	125	3687.5	3688	0.7	22	SP-SC	9		0.56	1.02	23	0.26	0.24	0.93	0.41	1.24	0.87	0.38	0.68
B-3	34.5	20	SPT	y	o2	1	1	1.2	1.00	1.00	24	35	125	4312.5	4313	0.7	17	SM	15		2.50	1.05	20	0.22	0.18	0.89	0.39	1.24	0.84	0.38	0.58

ATTACHMENT E.2

CLiq Liquefaction Results

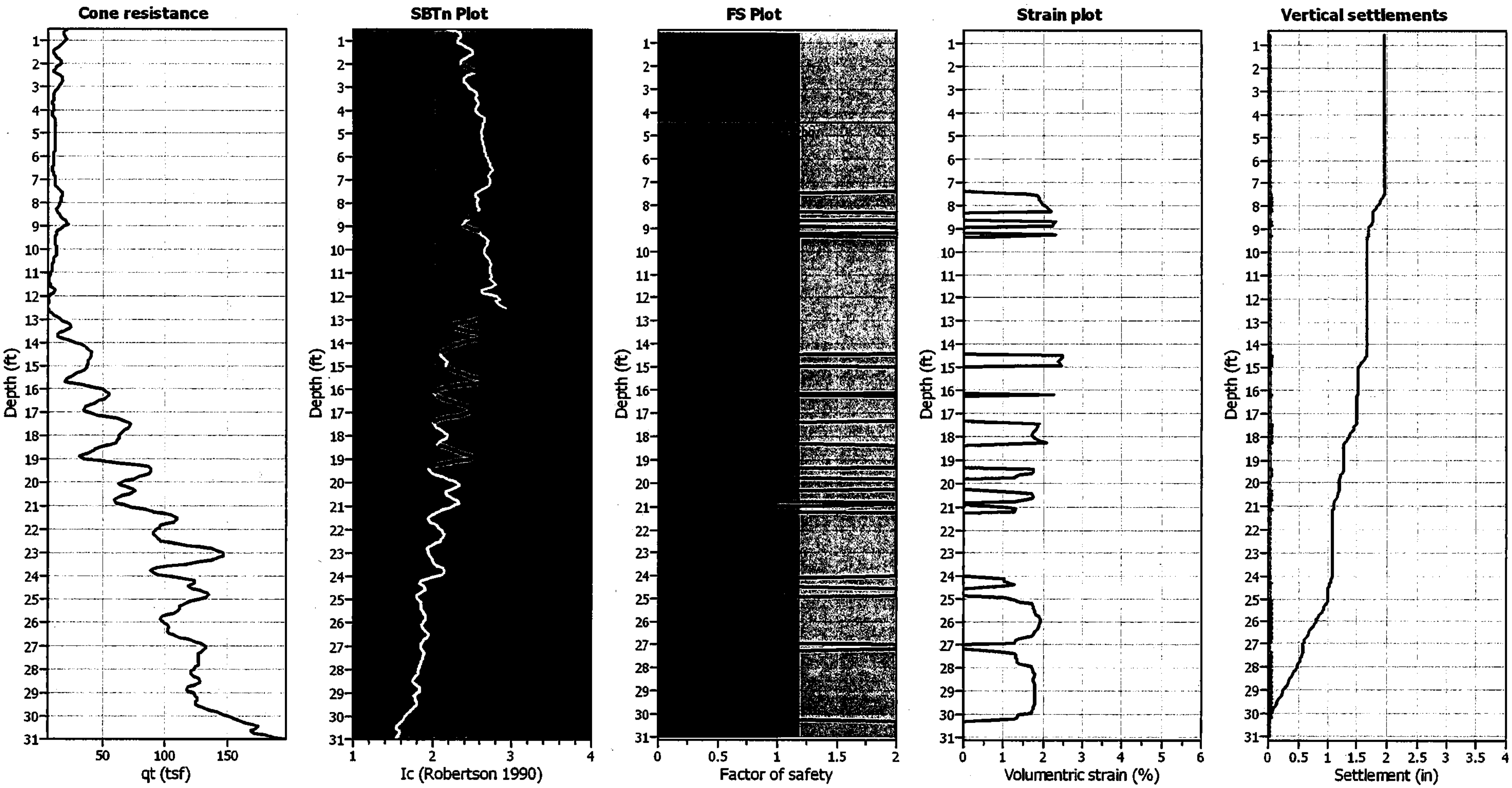
Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Estimation of post-earthquake settlements



Cone resistance

SBTn Plot

FS Plot

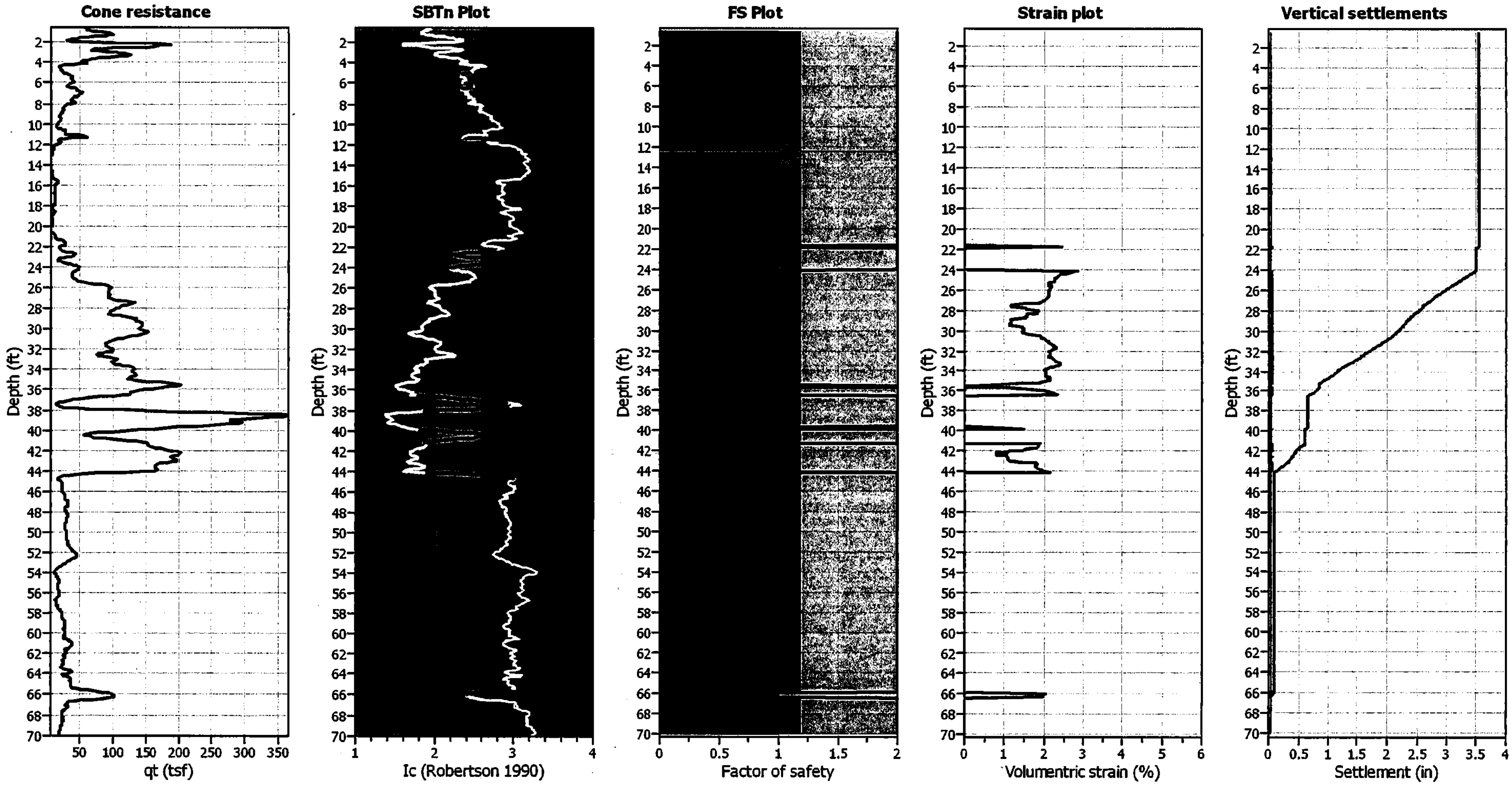
Strain plot

Vertical settlements

Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

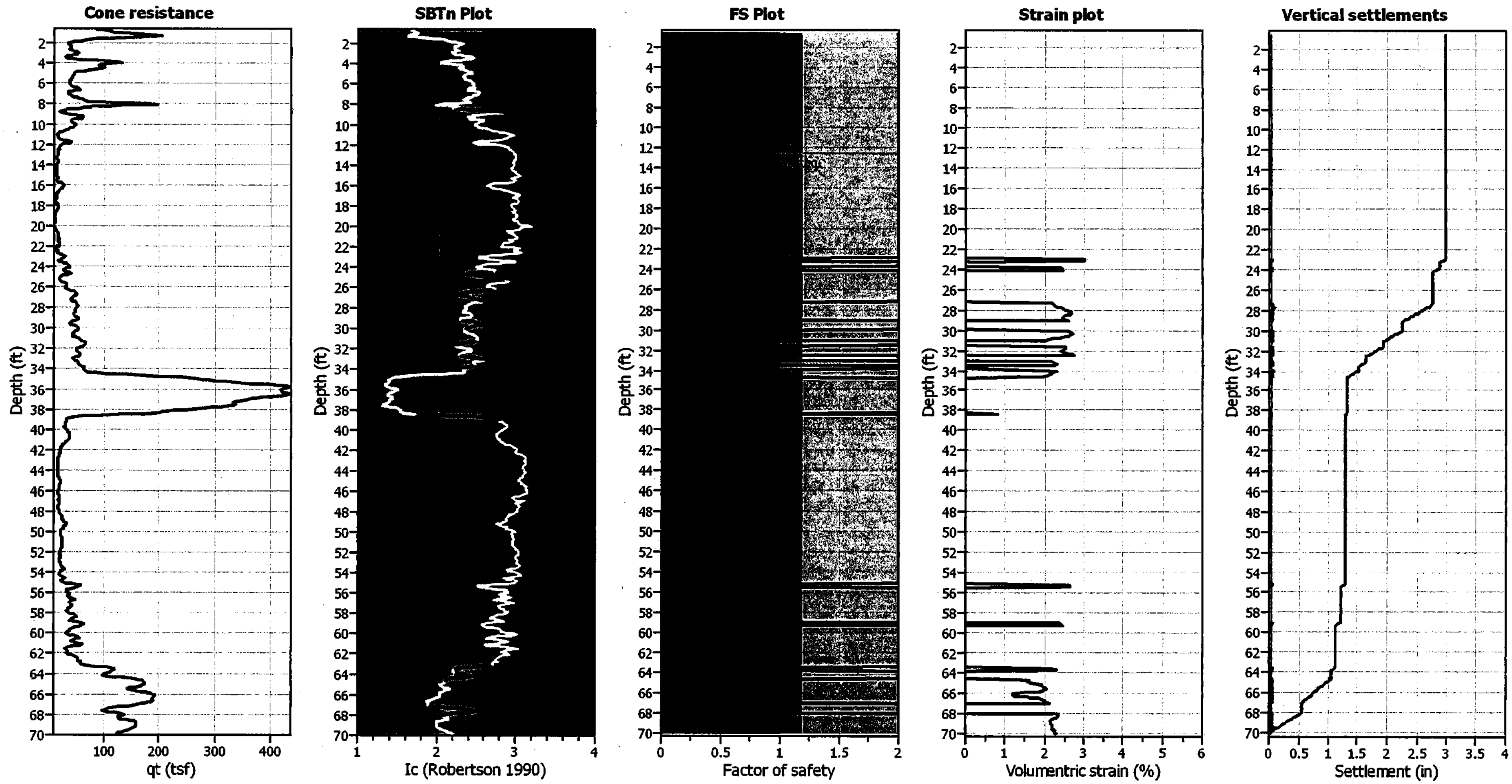
Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

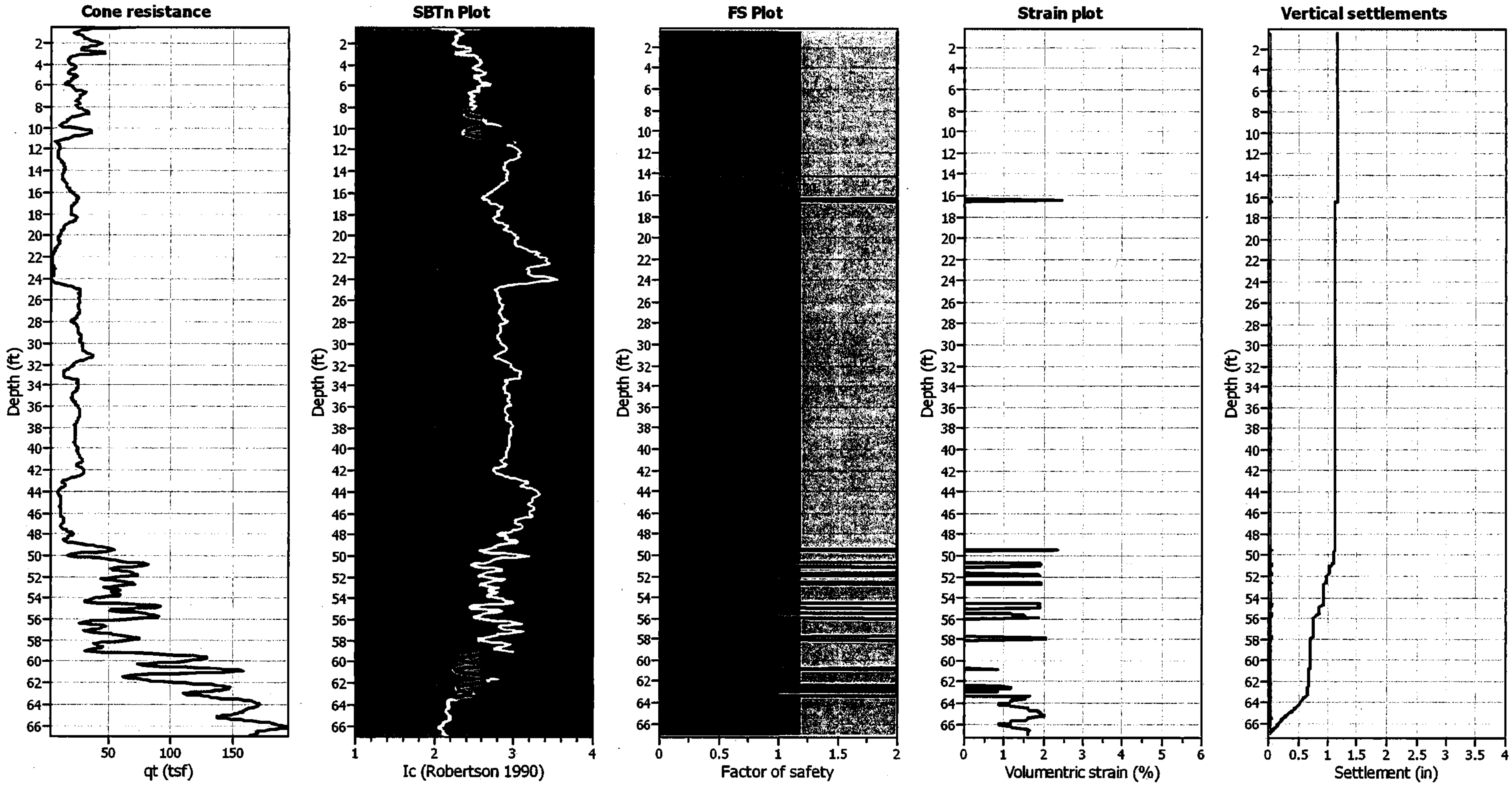
Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Estimation of post-earthquake settlements



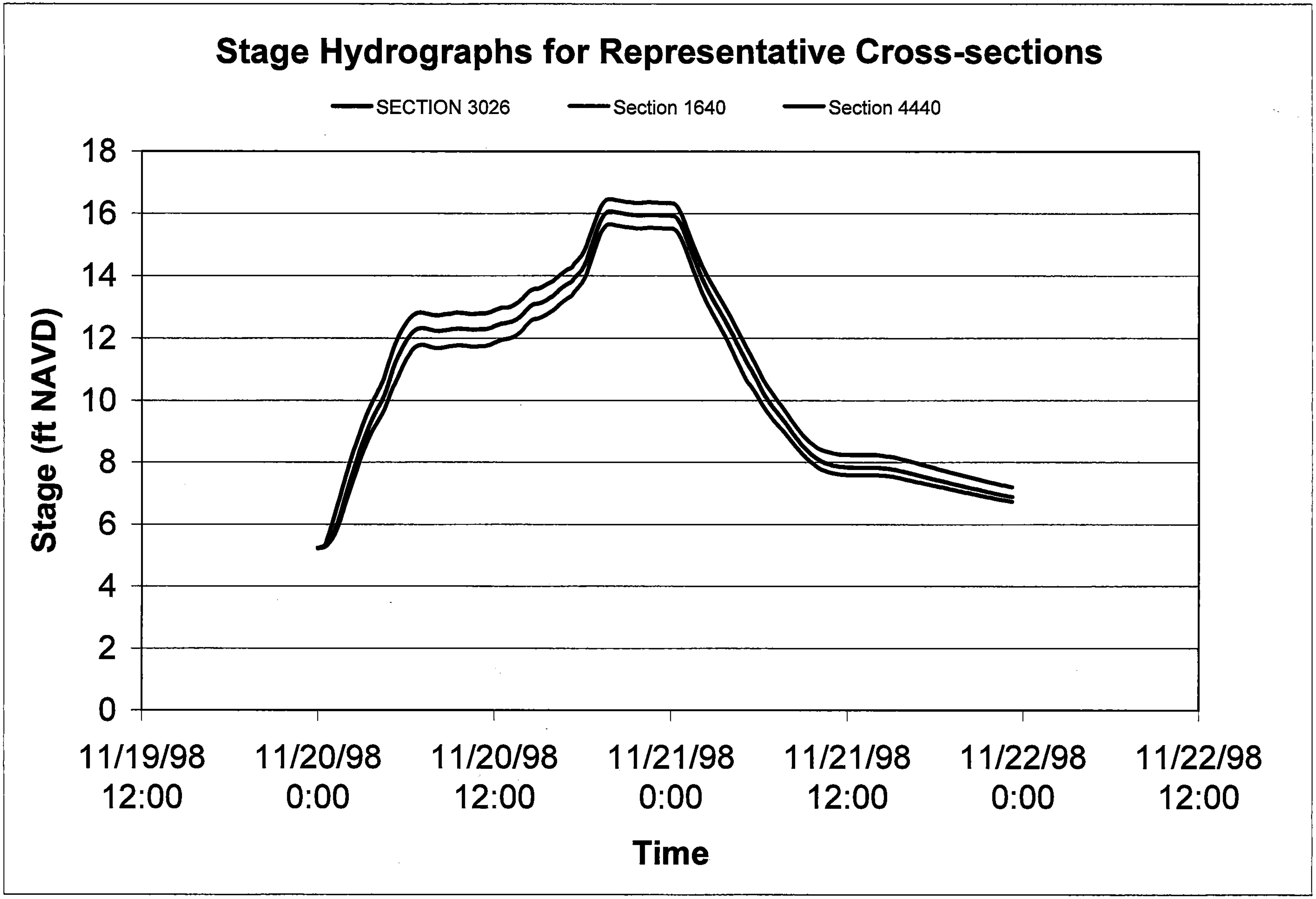
Abbreviations

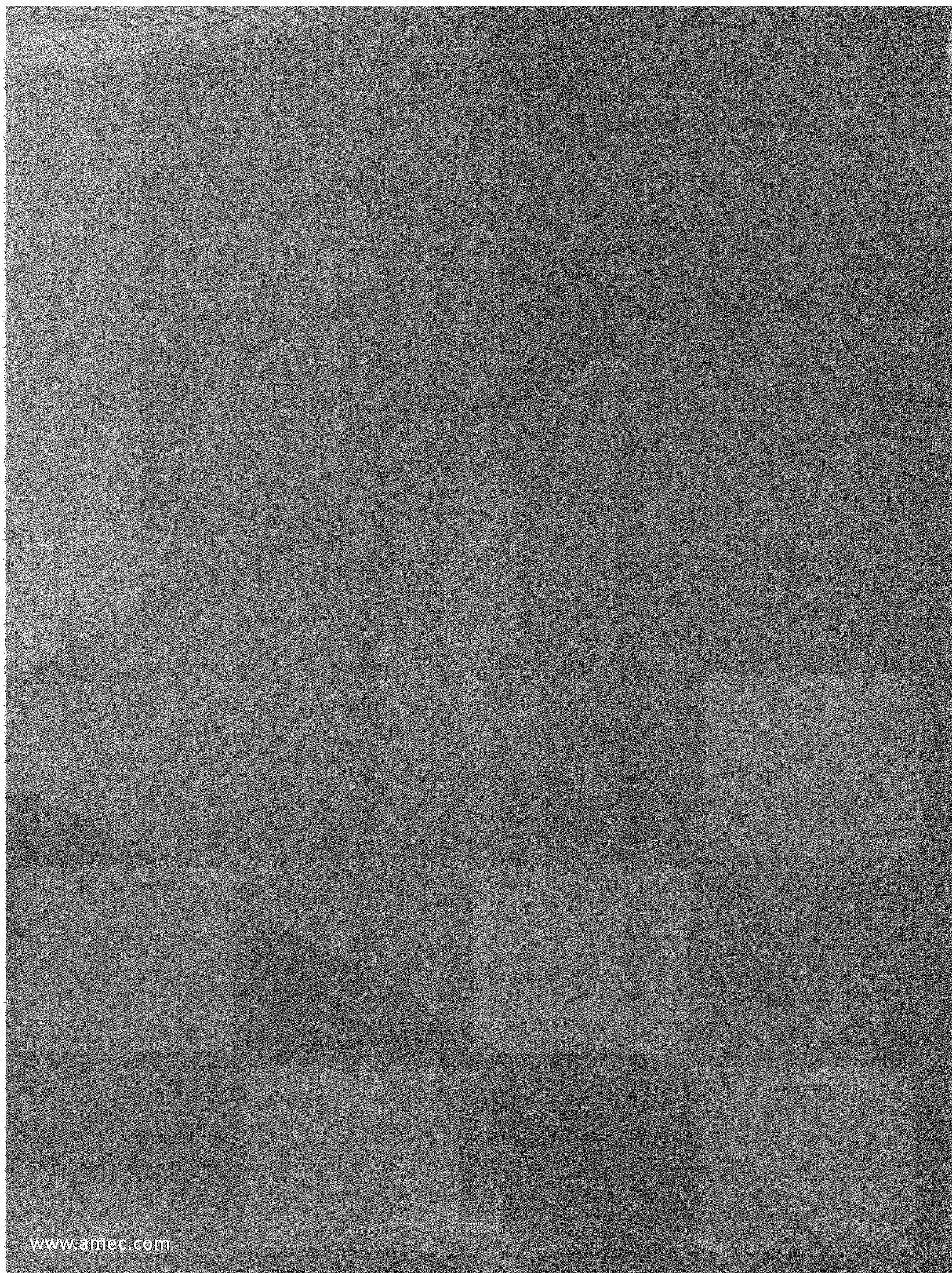
q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

APPENDIX F

APPENDIX F

Lower Penitencia Creek Design Flood Stage and Hydrographs


Stage Hydrographs for Representative Cross-sections
— SECTION 3026
— Section 1640
— Section 4440
Stage (ft NAVD)
0
2
4
6
8
10
12
14
16
18
11/19/98 12:00
11/20/98 0:00
11/20/98 12:00
11/21/98 0:00
11/21/98 12:00
11/22/98 0:00
11/22/98 12:00
Time



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